UNIVERSITY OF ALBERTA

LANDSLIDES AT THE ROCK GLACIER, HIGHWOOD PASS

BY

RODNEY P. MCAFFEE ©

A thesis submitted to the Faculty of Graduate Studies and Research in partial fulfillment of the requirements for the degree of Master of Science.

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ABSTRACT:

Rock Glacier Site is located 2 km north of Highwood Pass in Kananaskis Country, Alberta. Toppling and related slope movements at this site were investigated. Preliminary work and extensive field mapping produced an airphoto interpretation, bedrock geology map, and topographical map. Insitu field testing and laboratory tests indicated that the external force driving the toppling process was frost action and heaving of a weathered shale zone. A stability analysis of the slope included examining rock falls, rupture surface formation, and slides. Rock falls were determined to be a dominant form of slope movements. The development of rupture surfaces was found to occur when the amount of angular rotation of the toppling rock mass was between 13 and 16 degrees. The stability of slopes along fully developed rupture surfaces was found to be of concern. Finally, there were some implications for Highway 40, located at the toe of the slope.

ACKNOWLEDGEMENTS

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LIST OF SYMBOLS

Rs	Steep bedrock
R	Bedrock
$\frac{\text{Cs-A}}{\text{R}}$	Steep avalanched colluvium overlying bedrock
$\frac{\text{Ca}}{\text{R}}$	Apron of colluvium overlying bedrock
$\xrightarrow{39^\circ}$	surface slope in degrees
	Convex break in slope
	Scarp-sharp, length of downslope lines to give an indication of slope, line marks top of feature.
	Form line - raised feature.
	Talus slope.
	Line of Gully
80°	Inclined strata in degrees.
40°	Overturned strata in degrees.
	Certain geological boundary.
	Approximate geological boundary.
	Assumed geological boundary.

CHAPTER 1

Introduction

Rock slope movements have been studied in detail for years. Only recently has toppling been addressed in the literature. De Freitas and Watters (1973) were the earliest researchers to provide detailed descriptions of topples from the British Isles. Goodman and Bray (1976) then developed the first formal mathematical analysis to address the stability of toppling rock masses. This work on topples was on anaclinal slopes, where bedding dips into the slope. Tang (1986) and Eaton (1986) both identified topples on underdip cataclinal slopes in Kananaskis Country, Alberta. Underdip cataclinal slopes are characterized by bedding dipping more steeply but in the same direction as the slope. Hu (1991) then examined these topples as part of a broad investigation of rock slope movements in Kananaskis Country. The purpose of this thesis is to study in detail one particular site where topples have occurred, and are likely still occurring, on underdip cataclinal slopes. The location is 2 km north of Highwood Pass in Kananaskis Country. The slope, called *Rock Glacier Site*, is situated on the west flank of Mt. Rae immediately east of Highway 40, at an elevation of 2 300 m.

The work began with a literature review (chapter 2) and a preliminary site investigation (chapter 3). The literature review examined previous research on rock slope movements in Kananaskis Country (Hu 1991, Eaton 1986, Tang 1986, and Gardner 1980). The preliminary site investigation involved analyzing maps, airphotos, and climatic information on the area.

The next stage of research involved field mapping (chapter 4) and insitu field testing (chapter 5). The field work and insitu field testing was completed during the summer months of 1994 when access to the site was possible and the snow cover had mostly disappeared. Field mapping involved ground proofing the airphotos, mapping the bedrock geology, mapping the surficial geology, and establishing a topographical map and profile for the slope. Insitu field testing involved performing tests at *Rock Glacier Site*. A portable tilting table measured friction angles at the site, a rubber balloon apparatus measured insitu densities of weathered materials, and a standard rock hammer was used to estimate the uniaxial compressive strengths of rocks found on the slope.

To understand the behavior of the rocks and weathered material found at the site, laboratory tests were also completed (chapter 6). The tests included: density tests, grain size analysis, Atterberg Limits, swelling tests, calcium carbonate content tests, and frost susceptibility tests. These tests provided a more comprehensive understanding of the engineering properties of the rocks and weathered material at *Rock Glacier Site* which was then used in analyzing the stability of the slope.

The stability of the slope was addressed in terms of toppling, rock fall, and rock slide type movements (chapter 7). Water pressures in the slope were estimated and the processes controlling the development of toppling and rupture surface formations were investigated. A proposed sequence of processes which leads to slope movements at *Rock Glacier Site* has been developed which ties together the entire thesis. The final aspect of this research outlines the implications for Highway 40 with respect to future slope movements occurring at this site.

CHAPTER 2

Literature Review

2.1 Introduction

Mining and civil engineering projects frequently require the design of large slopes in rock. For mining engineering projects, these large slopes are usually cut into the rock in order to allow access to ore at great depths. For civil engineering projects, large rock slopes are usually already pre-existing and the engineer designs around the existing slopes. In both cases, the engineer either designs or modifies the slope to perform in an expected manner as related to the project. If it does not perform as expected, the slope can be said to have failed. However, natural slopes cannot be considered to fail because there is no expectation for performance of the slope. Therefore, "failures" on natural slopes are referred to as "slope movements".

One mechanism of slope movement of a rock slope is rotation, referred to as toppling. One of the earliest recognitions of natural slope movement by toppling was by De Freitas and Watters (1973). They gave a number of examples of topples from the British Isles. All of the slopes they examined had penetrative discontinuities dipping into the slope, termed anaclinal slopes (Cruden 1987a). Goodman and Bray (1976) divided toppling into common toppling and secondary toppling and investigated the mechanics of the process in detail. Common toppling occurs under the influence of gravity alone and secondary toppling is induced by loads transferred from sliding masses. However, their research only examined toppling movements on anaclinal slopes. Tang (1986) and Eaton (1986) both identified the presence of toppling movements of a different nature in Highwood Pass, in Kananaskis Country, Alberta. Tang (1986) was searching for field examples of toppling induced by external forces and Eaton (1986) was mapping rock slide hazards in Kananaskis Country. The topples which they found in Highwood Pass were situated in slopes exhibiting bedding which dipped more steeply but in the same direction as the slope. Slopes with this specific orientation with respect to bedding are referred to as underdip cataclinal slopes, according to the same terminology used by Cruden (1987a). Cruden (1989) extended the limits to common toppling by showing that it is kinematically possible for topples on underdip cataclinal slopes to occur using "the same assumptions as Goodman and Bray (1976) made to derive their kinematic criterion, for toppling on anaclinal slopes" (Cruden 1989: 738). This resulted in a need to further the study of toppling on underdip cataclinal slopes. As part of Hu's (1991) research, he examined in detail the processes of toppling in Kananaskis Country, including toppling on underdip cataclinal slopes.

2.2 Previous Research on Topples on Underdip Cataclinal Slopes

As part of Hu's (1991) research, he looked at toppling on underdip cataclinal slopes in Highwood Pass. Hu (1991) examined features and characteristics of each of the toppling movements identified. He also attempted to analyze the geological controls and explain the processes of toppling at each of the sites investigated. The topples on underdip cataclinal slopes identified by Hu (1991) were all located in shales and sandstones of Triassic age in the Spray River Formation. Hu (1991) found that the style of toppling was controlled by discontinuity spacing. To normalize the measured spacing of discontinuities, h, the spacing was divided by the bedding thickness, b, of the toppling mass, defined as the block ratio by De Freitas and Watters (1973). This ratio, h/b, was then used as a guide to predict the style of toppling that could be expected on other underdip cataclinal slopes. Hu (1991) recognized three styles of toppling on underdip cataclinal slopes. The three styles of toppling movements were block flexure topples, complex multiple block topples, and chevron topples. Block flexure topples were found to occur when the ratio of h/b was less than two. Complex multiple block topples were found to occur when the ratio of h/b was between 2 and 20 and sliding surfaces had developed within the topple. Chevron topples were found to develop with the same h/b ratio as complex multiple block topples when the angle of the sliding surface exceeded 35 degrees. Given these conditions and the geology and orientation of the underdip cataclinal slopes in Highwood Pass, Hu (1991) attempted to generalize the expected toppling style with location on the slope. Hu (1991) concluded that block flexure topples would dominate around the toes of the slopes, block topples or chevron topples would dominate in the middle of the slopes (depending on slope angles and bedding thickness), and block topples would dominate near the tops of the slopes. However, due to variations in geology, deviations from this generalization were observed.

Hu (1991) examined three topples located directly on *Rock Glacier Site*. Cruden and Hu (1994) discussed two of these topples in detail along with several other topples from the Highwood Pass, all occurring on underdip cataclinal slopes. Cruden and Hu (1994) identified the style of the topple near the top of *Rock Glacier Site* to be a multiple block topple, while the style of the topple near the toe of *Rock Glacier Site* was identified as a large block flexure topple. The oblique photo showing *Rock Glacier Site* clearly indicates that a large slope movement of some type has occurred (Fig. 2.1). Eaton (1986) suggested that toppling by creep was providing a continuous supply of debris to the large mass of colluvium at the toe of *Rock Glacier Site*. Hu (1991) only examined the small topples located on the slope individually and made no attempt to analyze the slope as a whole. As the large slope movement at *Rock Glacier Site* pre-dates historical records, it is not known whether there was one large single event or a multitude of small events responsible for the large mass of colluvium at the toe of the slope. Cruden (1987b) does suggests that it is unlikely that catastrophic movements have occurred on underdip cataclinal slopes. Cruden (1987b) qualifies this statement by adding that this inference is based on a limited amount of experience and case studies of toppling movements on underdip cataclinal slopes. The main purpose of this research is to build upon the work of others in the past by furthering the study of toppling movements located on underdip cataclinal slopes. Most importantly, this research will take Hu's (1991) work one step further and examine more closely the slope named *Rock Glacier Site* and attempt to determine its history of movements and future stability.

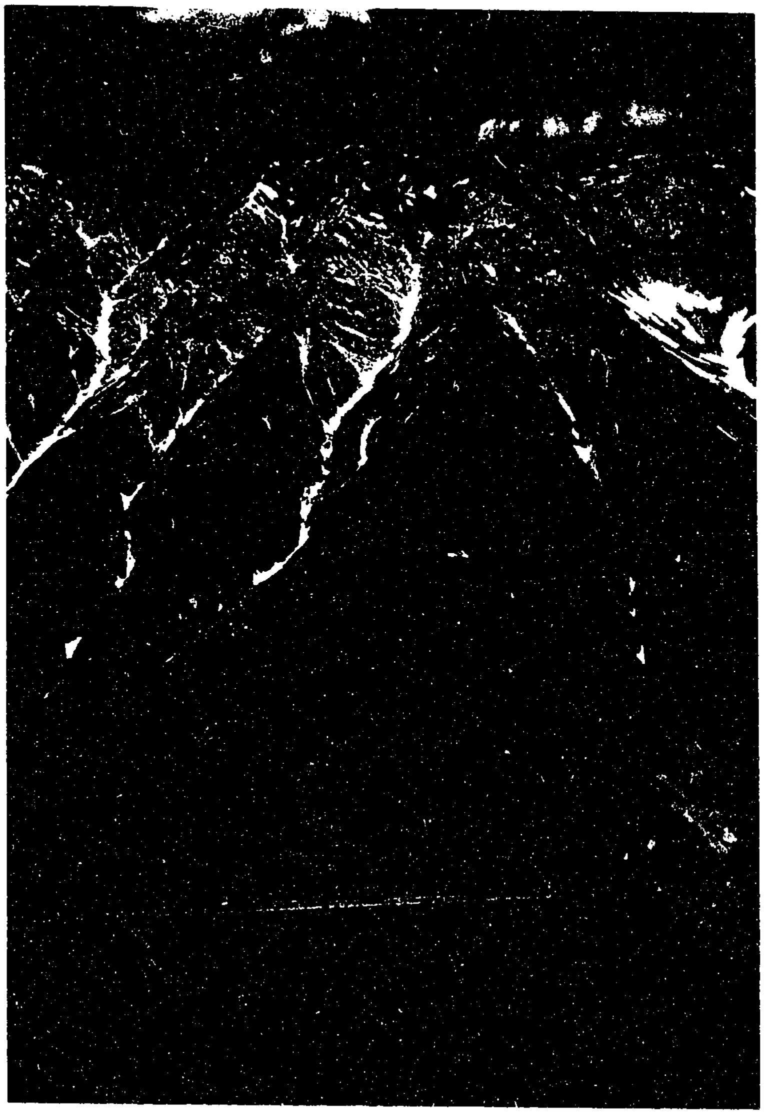


Figure 2.1: Oblique photo of *Rock Glacier Site* taken from Elk Range, looking towards the east.

2.3 Mechanisms of Toppling

As part of the history of *Rock Glacier Site*, we need to understand the mechanisms responsible for initiating these toppling movements. John (1970) suggested that the hydrostatic thrust of water in sub-vertical joint planes could lead to toppling of large rock blocks. Tang (1986), then added that the external forces responsible for toppling might include pressures induced from freezing processes or effects from the surcharge of accumulated ice and snow. Tang (1986) speculated that periglacial freeze-thaw action and ice wedging were the prime factors in causing the beds to topple. Schuster and Krizek (1978) state that frost action probably directly or indirectly accounts for more rock falls than all other factors combined. Perhaps this same mechanism for rock falls can be extended to include topples as Tang (1986) suggests. These toppling mechanisms will each be considered with respect to *Rock Glacier Site*.

2.4 Rock falls and Rock slides

The existence of other types of slope movements at *Rock Glacier Site* will be examined in chapter seven along with toppling type movements. The processes of rock falls and rock slides will be evaluated at the site and their significance with respect to past and future movements will be explored. Gardner (1980) describes the frequency, magnitude, and spatial distribution of rock falls and rock slides in the Highwood Pass. *Rock Glacier Site* lies nearly directly in the center of the study area investigated by Gardner (1980). Gardner (1980) states that field observations indicate that low magnitude / high frequency rock falls and / or rock slides are likely to be the primary processes responsible for the massive debris slopes in Highwood Pass. However, at an average frequency of 0.83 events per hour of observation, low magnitude / high frequency rock falls and / or rock slides cannot account for the massive debris slopes present in the area (Gardner 1980). This suggests that low frequency / high magnitude events must transport greater volumes of material than low magnitude / high frequency events unless higher rates of these low magnitude events occurred in the past, as is suggested by landforms in the area (Gardner 1980). Field observations and analysis of the geometry of the debris slopes at *Rock Glacier Site* will attempt to determine the role of rock falls and rock slides. Furthermore, it will be determined whether low magnitude / high frequency events or high magnitude / low frequency events are responsible for past movements and which type of event is expected to occur in the future at *Rock Glacier Site*.

2.5 Developments in Insitu Testing

In order to perform an accurate stability analysis of the slope, the friction angles of the rocks which make up the slope need to be determined. As part of Hu's (1991) research, a portable tilting table was constructed and tested. The results, when compared with standard laboratory tests, indicated that the portable tilting table, used in the field, measured the friction angle along discontinuities that were very close to the basic friction angle of the rock being tested (Hu 1991). Tests on sandstone samples indicated that the minimum sliding angle determined was close to the upper bound of basic friction angles obtained from laboratory tests (Hu and Cruden 1992b). Thus, the portable tilting table developed by Hu (1991) was used as part of this research program to speed up what might otherwise have been a time consuming process of sample preparation and laboratory testing.

2.6 Conclusions

The purpose of this research project was to build upon previous work which has examined the kinematics of topples (Goodman and Bray 1976) in general and the work done specifically on topples in Highwood Pass (Hu 1991, Eaton 1986, and Tang 1986). Specifically, this research is a continuation of Hu's (1991) work related to topples on underdip cataclinal slopes located at *Rock Glacier Site* in Highwood Pass. In chapter seven, individual topples are characterized and analyzed, the history of slope movements considered, the mechanisms of toppling movements explored, and the future stability of the slope evaluated. The role of rock falls and rock slides will also be examined in chapter seven and an attempt will be made to determine whether low magnitude / high frequency events or high magnitude / low frequency events are responsible for past slope movements and which type of event is expected to occur in future movements.

CHAPTER 3

PRELIMINARY SITE INVESTIGATION

3.1 Introduction

Before starting the field mapping and the actual site investigations, a significant amount of information can be obtained through researching existing information and through airphoto interpretations. The existing information consists of maps (bedrock, surficial, and topographical), climatic records, and aerial photographs. Maps can indicate lithology, structural features, and surficial features. Climatic data can indicate the environmental conditions in the area. Environmental conditions can be used to estimate the dominant form of weathering and as an indicator of the degree of weathering that can be expected in the area. Aerial photographs can be used to examine topography, slope movements, drainage characteristics, gullies, vegetation, and land uses in an area. In remote areas, aerial photographs can be used to examine large areas of land quickly and easily. For this project, aerial photographs were essential in the preparation of the actual site investigation planned for the very short summer field season associated with field mapping in the high mountain areas of the Canadian Rocky Mountains.

3.2 Location of Site

The site is east of Highway 40 two kilometres north of Highwood Pass in Peter Lougheed Provincial Park (Fig. 3.1). Peter Lougheed Provincial Park is in turn located in Kananaskis Country, in the Front Ranges of the Canadian Rockies in Alberta. The slope is situated east of Highway 40 with the toe of the slope immediately adjacent to the Highway (Fig. 3.2). The slope is named *Rock Glacier Site* and is designated by park authorities as a tourist stop. A small hiking trail with several informative signs detailing information regarding the history of the colluvium, the local wildlife, and types of

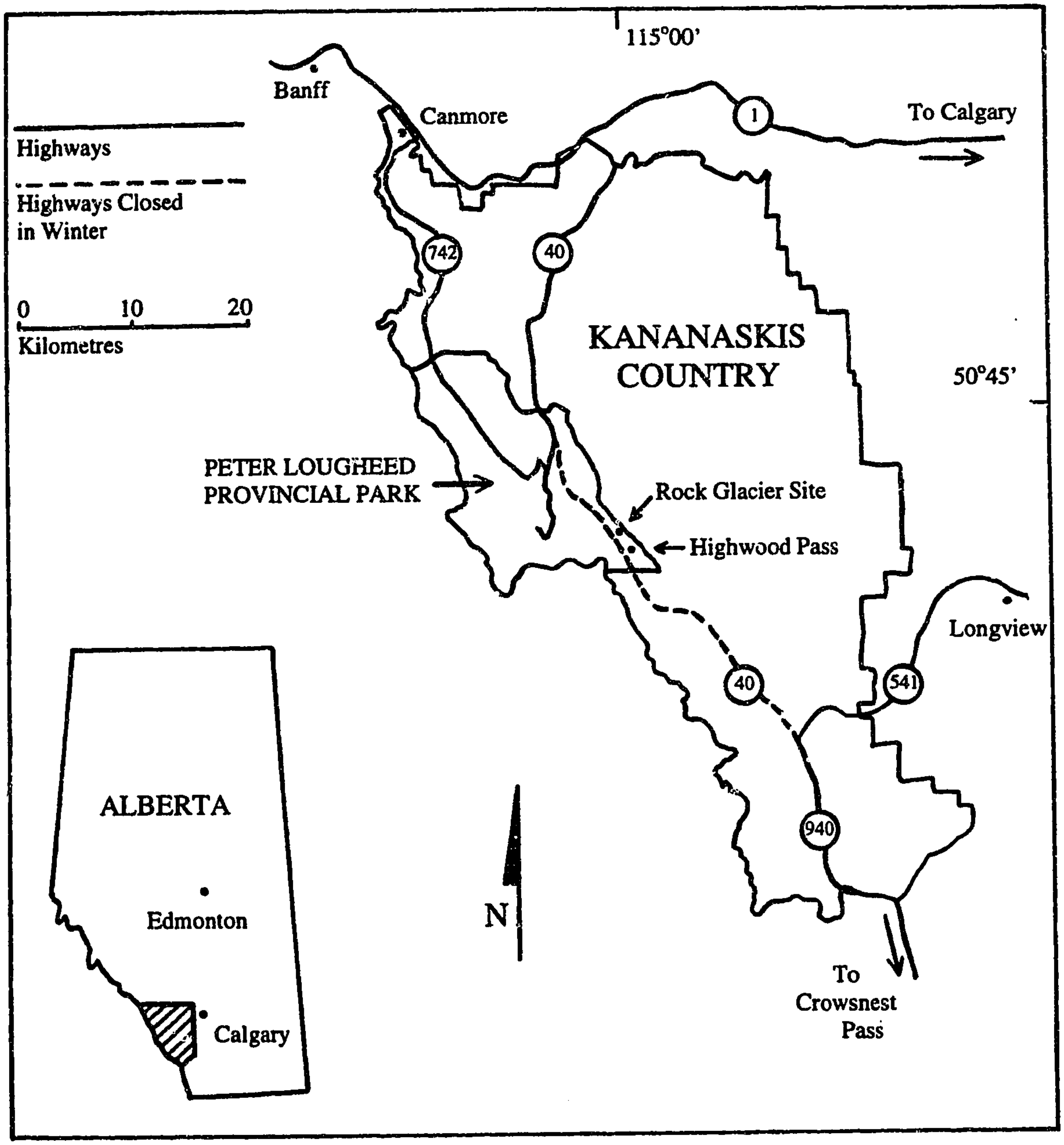


Figure 3.1: Map Showing Location of *Rock Glacier Site* in Kananaskis Country, Alberta.

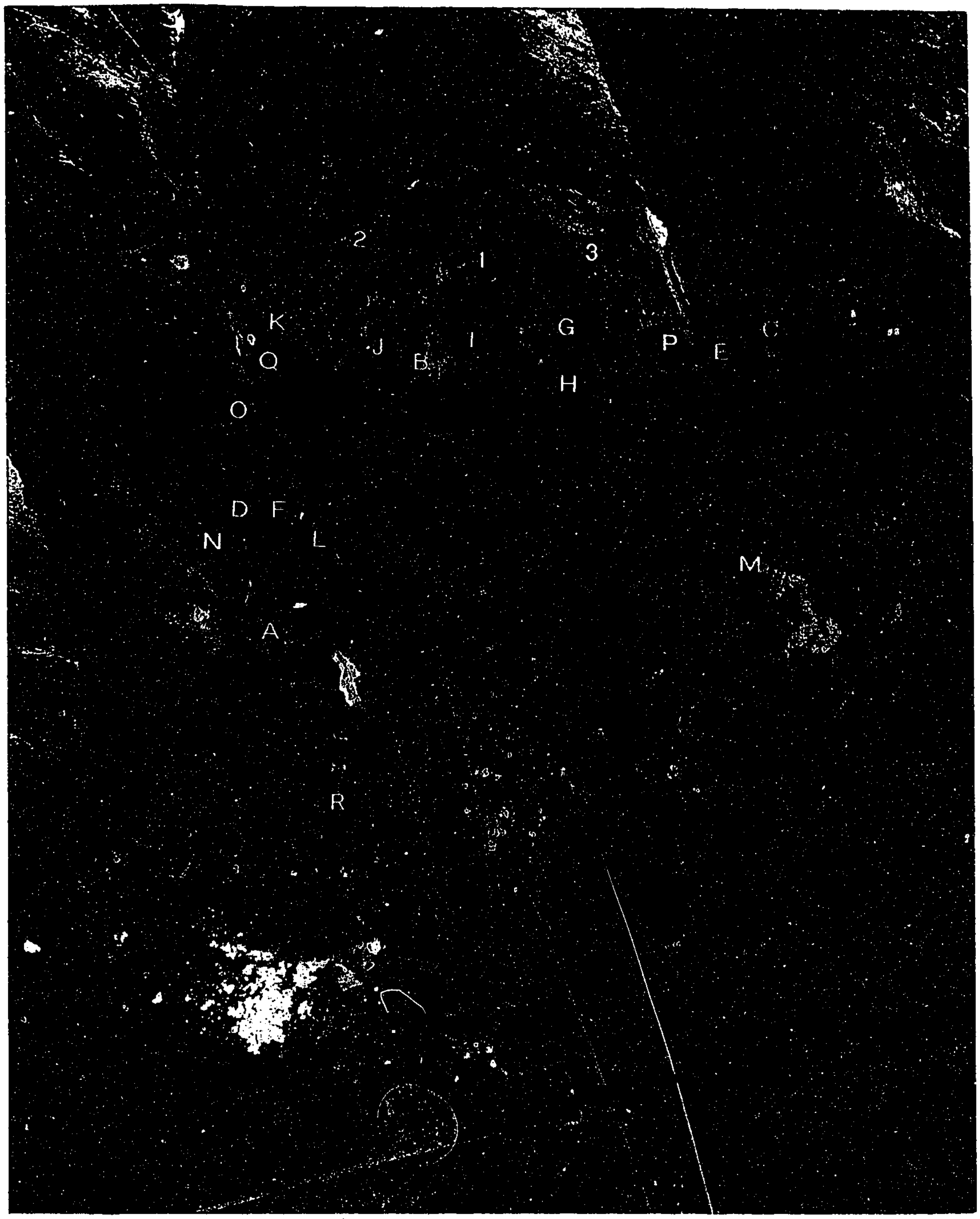


Figure 3.2: *Rock Glacier Site* with the 3 test sites indicated in the weathered shale zone (labeled as 1, 2, and 3) and location of all 20 topples identified at *Rock Glacier Site* labeled from "A" through "R" (1:5 000 scale). Bearing along highway is 140 degrees.

vegetation in the area provide for a popular tourist attraction in Peter Lougheed Provincial Park.

The site is on NTS map sheets 82J/10 and 82J/11 (1:50 000 scale). The latitude of the site is 50 36' 53", while the longitude of the site is 115 00' 10". The topographical contours on these maps were used to estimate the mean elevation above sea level of the site, at 2 300 m.

Rock glaciers, by definition, are large masses of angular rock and finer material with interstitial ice or an ice core (Bates and Jackson 1987). Bates and Jackson (1987) added that rock glaciers situated on slopes, are prone to creep as the ice within the mass deforms in a plastic manner under the influence of gravity, similar to small valley glaciers. Rock glaciers occur in areas of permafrost and are derived from a cirque wall or other steep cliff (Bates and Jackson 1987). The description as a rock glacier is, in fact, an inappropriate name for this slope because there is no evidence of permanent ice within the colluvium amassed at the toe of the slope at *Rock Glacier Site*. In fact, what is likely to be occurring is the slow movement of the displaced mass down slope with no aid from permanent internal ice. A more detailed discussion of the slope's structure and movements is in chapters four and seven.

3.3 History of Investigation

NTS map sheets 82J/10 and 82J/11 were used to initially locate *Rock Glacier Site*. However, at a scale of 1:50 000, only 30.48 m (100 ft) elevation contours were available. To obtain better resolution with respect to elevation contours, digital base 1:20 000 scale maps of the same area were consulted (Alberta Environmental Protection 1992). These maps provided better resolution of topography with a contour interval of 20 m (Fig. 3.3).

Re: Copyright permission

Copyright permission was granted for use of the 1: 20 000 scale Provincial Digital Base contour map being used in the thesis - Landslides at the Rock Glacier, Highwood Pass (Figure 3.3). Permission was given verbally, in accordance with the policy at Alberta Environmental Protection, by Mr. Mike Wells in February 1995. Mr. Mike Wells can be reached at Alberta Environmental Protection at (403) 427 - 5955.

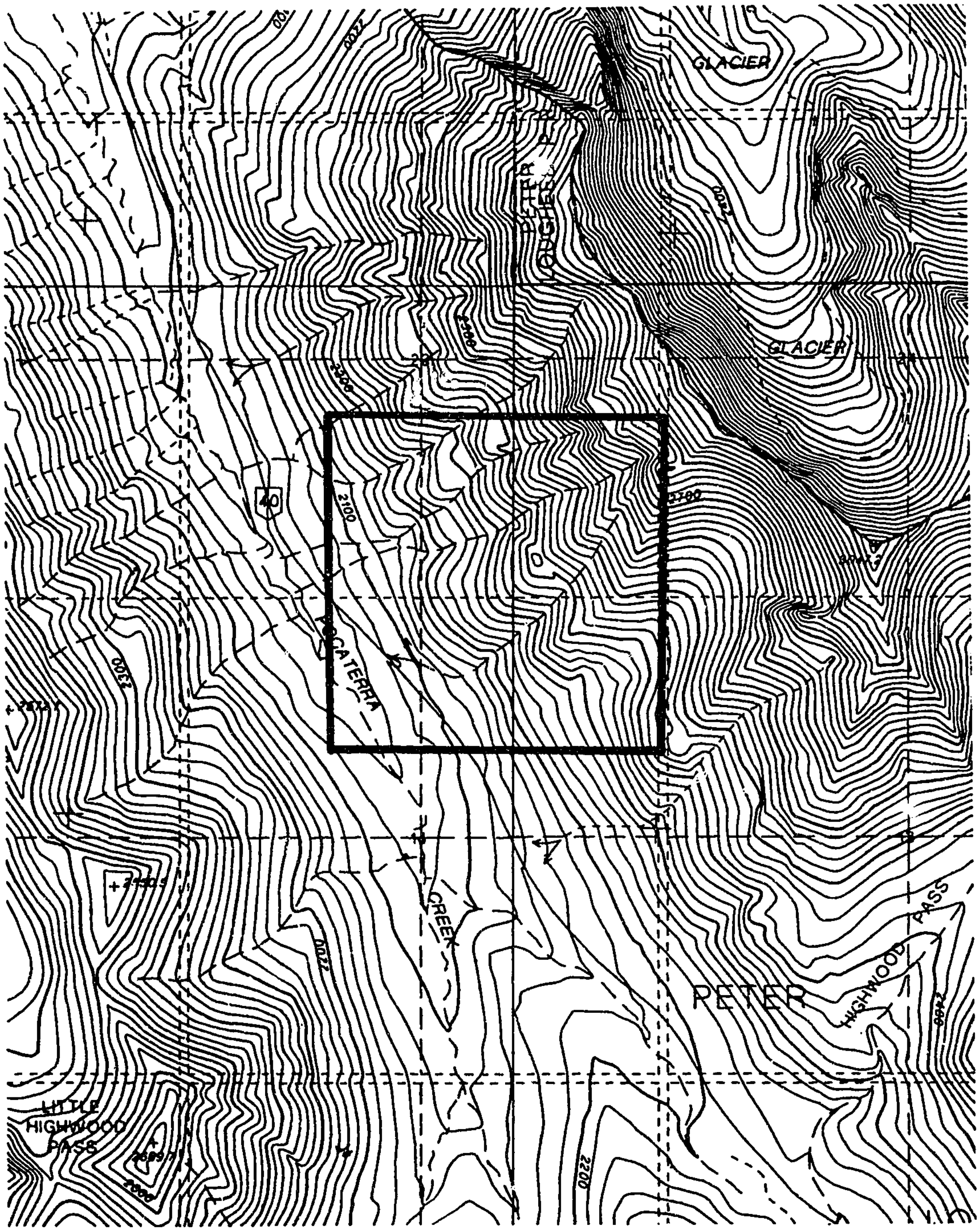


Figure 3.3: 1:20 000 scale topographical map with 20 m elevation contours and location of *Rock Glacier Site* outlined (Alberta Environmental Protection 1992).

For an overall look at the bedrock geology and the surficial deposits in the Highwood Pass, Surficial and Bedrock Geology Maps of the Alberta Foothills and Rocky Mountains were examined (L.A. Bayrock and T.H.F. Reimchen 1975). To obtain more detailed information about the bedrock geology in the area, the Geological Map of Peter Lougheed Provincial Park was consulted (McMechan 1989). At a scale of 1:100 000, this map provided useful information about the lithology and age of the bedrock as well as information about thrust and transverse faults in the area. *Rock Glacier Site* is marked as a landslide on the Geological Map of Peter Lougheed Provincial Park (McMechan 1989).

Using the maps mentioned above, an overall sense of the area to be investigated was achieved. The bedrock was likely to be sedimentary, probably sandstone, siltstone, and shale. The lower part of the slope, in the Fernie Formation, is of Jurassic age, and the upper part of the slope positioned in the Spray River Formation is of Triassic Age. Continuing further up the slope, Permian chert and dolomite would be expected. The surficial deposits in the area, as shown by the surficial geology map, were colluvium, alluvial fans, and aprons.

Hu (1991) studied the area of Highwood Pass in detail. As part of his research, he analyzed 16 topples on cataclinal underdip slopes in the area of *Rock Glacier Site*. All 16 topples being located along the same north-west south-east trending ridge as *Rock Glacier Site* is located. Of the 16 topples investigated, three were located directly on *Rock Glacier Site*. Hu's (1991) research provided information on deformation styles, discontinuities formed by toppling, lithology, strength of rock masses, and topography. The stratigraphic profile given by Hu (1991) indicates shales of the Fernie Formation at the toe of the slopes along the north-west south-east trending ridge, sandstones and shales of the Spray River Formation in the middle of the slopes, and quartzites from the Rocky Mountain Formation further up the slopes. All the topples identified by Hu (1991) were located in the Spray River Formation, in the middle section of the slopes. Hu (1991) indicates that bedding on these slopes dips steeply to the west in the same direction as the slope. Therefore, since the bedding dips in the same direction as the slope, but at a steeper angle, the slopes along this ridge are cataclinal underdip slopes (Cruden 1989).

The information provided by the various maps consulted and the research done by Hu (1991) provided a basic knowledge of the area. This basic knowledge was used to enhance the airphoto interpretation of *Rock Glacier Site*.

3.4 Climate and Its Effect

Climate plays an important role in the rate and degree at which the weathering process will alter exposed rock. Intense weathering, or prolonged exposure to weathering, of a rock mass will significantly reduce its overall strength. Thus, weathering of a rock mass situated on a steep slope may decrease its stability. To understand the weathering processes, we need to first determine the climatic conditions at *Rock Glacier Site* in Highwood Pass.

Due to the variability of weather in mountainous areas, it would be ideal to have climatic data which was directly recorded in Highwood Pass. However, due to the winter closure of Highway 40 and the remoteness of the area (Fig. 3.1), there are no climatic records available. There is however climatic data available from other locations in Kananaskis Country. The University of Calgary has a field station near Barrier Lake, located approximately 50 km north of Highwood Pass. At the field station, Environment Canada has accumulated complete climatic records since 1939. Based on a 30 year average, the mean annual temperature is 3.1 °C and the mean annual precipitation is 638.1 mm (Environment Canada 1993). Climatic records further indicate that from the months of November through March, the mean daily temperatures are below 0 °C. The average number of degree-days below 0 °C each year is 999 (Environment Canada 1993). The number of degree-days below 0 °C each year is calculated by summing the average daily temperatures of all the days each year which are below 0 °C. The climatic records also indicate that many freeze-thaw cycles may occur in this area each year. For 1990, there were 127 days where both the minimum daily temperatures were below 0 °C and the maximum daily temperatures were above freezing (Environment Canada 1993). The precipitation which falls in the area is highest during the months of May and June and varies from a monthly low of 29.3 mm in November to a monthly high of 85.0 mm in June (Environment Canada 1993).

The University of Calgary Field Station is located at an elevation of 1 391 m. *Rock Glacier Site* in Highwood Pass is located at 2 300 m. The difference in elevation between the two sites can result in a marked change in climate. To better estimate the climate at Highwood Pass we need information from a site at a higher elevation. During the summer months of June, July, and August, Environment Canada records climatic data at a site located east of Upper Kananaskis Lake at an elevation of 2 073 m. This site is located approximately 5 km away from Highwood Pass. Therefore, we can compare the differences in mean monthly temperatures and precipitation between the two locations during the summer months and extrapolate the differences to the other nine months of the year.

The site at the higher elevation is on average 2.9 °C cooler during each of the three summer months (Environment Canada 1993). The drop in temperature with increased elevation, the lapse rate, or thermal gradient, gives the rate of change in temperature with a change in elevation above sea level. The Glossary of Geology gives the lapse rate as -0.6 °C for every 100 m increase in elevation (Bates and Jackson 1987). According to the lapse rate, the temperature at the higher elevated station, near Upper Kananaskis Lakes, should have ideally been 4.1 °C cooler than the temperatures at the lower elevated station, at the University of Calgary Field Station. If we consider the rate of decrease in temperature between the two climate stations discussed here and extrapolate the difference to *Rock Glacier Site* then the mean annual temperature at *Rock Glacier Site* should be -0.8 °C.

When the average monthly precipitation between the two sites is compared, the site near Upper Kananaskis Lake measured 2.0 mm more precipitation than the site at the University of Calgary Field Station (Environment Canada 1993). If the monthly average is 2.0 mm higher then we can assume that the annual average is 24.0 mm higher. Thus the mean annual precipitation at the site near Upper Kananaskis Lake would be 662.1 mm. Since precipitation is not as much a function of altitude, we assume the precipitation to be the same at Upper Kananaskis Lake and *Rock Glacier Site* due to their close proximity.

Having estimated the climatic conditions at *Rock Glacier Site*, we can now determine the likely form of weathering and the likely degree of this weathering. Peltier (1950) developed criteria for the form of weathering and type of weathering by considering the mean annual temperature and precipitation. As the mean annual temperature and precipitation at *Rock Glacier Site* was established as -0.8 °C (30.6 °F) and 662.1 mm (26.1 inches), using Peltier's (1950) criterion, we would expect weak to moderate chemical weathering and slight to moderate mechanical weathering at *Rock Glacier Site*. Peltier's (1950) criteria also predicts weak to moderate frost action and moderate mass movements to be probable for these climatic conditions. The importance of frost action in terms of mechanical weathering in Highwood Pass is further supported by Hu and Cruden's (1992a) description of the area as being a periglacial environment which is susceptible to freeze-thaw cycles during the spring and fall seasons. There is a need for more detailed information on the number of freeze-thaw cycles which are occurring at *Rock Glacier Site*.

3.5 Airphoto Interpretation

Several series of aerial photographs have been flown over the Highwood Pass (Table 3.1). However, *Rock Glacier Site* was consistently positioned near the edge of all the 1:20 000 scale airphotos. Being positioned near the edge of the airphotos caused significant distortion of the image, which greatly reduced the detail. One series of

Project Number	Date	Scale	Emulsion
93 - 130	1993 - 00 - 00	1 : 40 000	PAN 150
92 - 098	1992 - 09 - 22	1 : 20 000	IR 2424
91 - 210	1991 - 08 - 08	1 : 20 000	IR 2424
88 - 131A 82J	1988 - 07 - 20	1 : 130M	PAN 50
88 - 216	1988 - 00 - 00	1 : 40 000	PAN 150
S84 - 077**	1984 - 07 - 17	1 : 10 000	PAN 2405
84 - 58	1984 - 08 - 09	1 : 20 000	CNEG 2445*
F83 - 051A I	1983 - 10 - 07	1 : 25 000	PAN 2405
82 - 176	1982 - 07 - 23	1 : 20 000	CNEG 2445*
82 - 52	1982 - 00 - 00	1 : 60 000	PAN 2405
S80 - 012	1980 - 09 - 26	1 : 25 000	PAN 2405
79 - 144	1979 - 10 - 03	1 : 30 000	CPOS 2443*
76 - 51C	1976 - 09 - 19	1 : 12 000	IR 2424
76 - 51D	1976 - 06 - 18	1 : 12 000	PAN 2405
73 - 162	1973 - 07 - 02	1 : 12 000	PAN 2405
72 - 65	1972 - 06 - 28	1 : 21 120	IR 2424
69 - 252	1969 - 05 - 08	1 : 12 000	PAN 2405
69 - 229	1969 - 05 - 08	1 : 24 000	PAN 2405
58 - 82J	1958 - 00 - 00	1 : 15 840	SUPER XX
49 - 82J	1949 - 00 - 00	1 : 40 000	SUPER XX

**** Airphotos selected for this project.**

Table 3.1: Airphotos flown over Highwood Pass (available through Maps Alberta).

airphotos flown at 1:40 000 scale did have the site favorably situated. However, the small scale of the airphotos meant that they would have to be enlarged eight times to provide the adequate detail needed. Magnifying the airphotos to this degree would have created significant distortion of the image. This distortion would have made viewing in stereo more difficult. The aerial photographs that were finally decided upon were at a scale of 1:10 000 and were flown along Highway 40 in 1984. Having followed Highway 40, not a straight flight line, these aerial photographs would be difficult to view in stereo. However, in the area of *Rock Glacier Site*, the flight path was close to a straight line and the site was located near the center of the airphotos. To further increase the detail, the airphotos were enlarged to 1:5 000 scale, a magnification factor of two. Using these aerial photographs, a detailed airphoto interpretation was performed.

The results of the airphoto interpretation are shown in Figure 3.4. Symbols and descriptions used for the airphoto interpretation were taken from Cruden and Thomson (1987). Terminology for the slope itself follows the suggested nomenclature for landslides as given by the IAEG Commission on Landslides and Other Mass Movements (1990). The terrain at *Rock Glacier Site* was divided into two units, colluvium and bedrock. The main scarp from the toppling events in the center of the slope and the main scarps in each of the upper gullies were easily identified on the aerial photographs. The talus slopes and direction of active debris movements were also easy to trace. The main zone of accumulation of material on the slope was estimated to cover an area of 125 000 m^2. The drainage pattern was found to be dendritic. All surface water was observed to flow into the gullies on either side of the slope and then down the slope where it flows into Pocatera Creek which flows towards the north in the valley bottom. The tree line is midway up the slope at an elevation of about 2 600 m. Below tree line, there is moderate to thick coniferous vegetation where the colluvium is not actively accumulating. At the tree line, there are some grassy slopes and sparse stunted trees. Evidence of snow avalanches can be seen at the bottom of each gully. The avalanche path is denoted by downed trees and younger trees growing where the

snow has run out during past avalanches. The three toppling events studied by Hu (1991), at this site, can also be seen on the airphoto (Fig. 3.2).

The airphoto interpretation provides valuable information about the site. By knowing the areas of exposed bedrock and the type of terrain present, the field mapping plan can be established before the actual site investigation begins. The aerial photographs provide information about toppling event locations, and areas of active debris accumulation also referred to as the zone of accumulation. Most importantly, the airphoto interpretation provides an overview of the entire slope being studied. Due to the large size of the slope, a ground based site investigation does not provide an adequate overview of the entire site. Furthermore, the aerial photographs can be used as a guide in constructing the regional geology maps and the profiles needed to analyze the overall stability of the slope.

3.6 Conclusions

From the existing information available, a significant amount of data pertaining to *Rock Glacier Site* was obtained. The exact location and elevation above sea level of the site was accurately found using various maps. Detailed information about lithology, surficial deposits, and topography was also determined with maps. The climate at the site and its probable effects with respect to weathering were estimated. Finally, aerial photographs were used to analyze the terrain at the site. The airphoto interpretation provided enough information to locate specific areas of interest and provided a basis for planning the field mapping and site investigation program. The aerial photographs were also used as an aid in constructing the regional geology map and in constructing the slope profiles.

CHAPTER 4

FIELD MAPPING

4.1 Introduction

The remote location of this site, its high altitude, and the closure of Highway 40 during the winter months restricted all field mapping to the summer months. Each year, Highway 40 is closed from King Creek to Highwood House between December 1 and June 15 (Fig. 3.1). The closure is necessary to protect motorists against frequent snow avalanches which occur along the highway and to provide a sanctuary for the wildlife in the area. Taking advantage of the warmer weather during the summer months, the entire field mapping program planned for the site was completed between June 16, 1994 and August 31, 1994.

The field mapping program planned for the site investigation at *Rock Glacier Site* was broken into four main parts. The first part of the field program involved ground proofing the airphoto interpretation. Ground proofing the airphoto interpretation was necessary to ensure that the features observed on the aerial photographs were the same as those observed on the ground. Ground proofing also allowed small or shadow obscured features to be examined for the first time. The second step in the field mapping program was to map the bedrock geology of the site. Mapping the bedrock geology involved identifying lithologies, measuring orientations of discontinuities, and measuring the spacing between discontinuities. The third step in the field mapping program involved mapping the surficial geology. The surficial geology provided information regarding slope movements, activity of movements, and volume of movements. The final step in the field mapping program was to determine the topography of the slope. Topographical profiles down the center of the slope and each of the adjacent gullies provided information essential for a stability analysis and for calculating the volumes of colluvium associated with the slope movements.

4.2 Ground Proofing Airphotos

The first stage of field mapping included a general reconnaissance of the area. The principal purpose of this general reconnaissance was to become familiar with the terrain and ground proof the airphotos. Ground proofing the airphotos ensured that all of the features determined through the airphoto interpretation were indeed accurate. Ground proofing the airphotos also allowed close examination of certain features that were either too small to be seen clearly or obscured by shadows on the aerial photographs. The general reconnaissance provided an overview of the topography and geology of the entire site and also ensured that the planned field mapping program was suitable.

The general reconnaissance also provided information about activity of debris movements and about activity of avalanches in the lower gullies. The presence of lichen on the debris on the talus slopes was a clear indicator of inactivity for a significant period of time. Conversely, the absence of lichen cover on debris was an indicator of recent movements or accumulation of debris. Further discussion regarding lichen cover on debris and the activity of slope movements will be covered with surficial geology. Ground proofing the lower gullies bordering each side of the slope provided proof of previous snow avalanches, debris flows, or slush avalanches. In fact, all the valley margins in the Rocky Mountains are locally prone to periodic debris flows (Jackson 1987). In the lower gullies there were many fallen trees, the majority of which had fallen in the down slope direction. There was also abundant scaring on trees still standing on the up slope side of the tree trunks. Some trees even had small rocks embedded into the up slope side of the tree trunks. These observations confirmed that frequent snow avalanches with some debris, debris flows, or slush avalanches do indeed travel down the lower gullies bordering each side of the slope.

4.3 Bedrock Geology (Stratigraphy)

The geological features of Peter Lougheed Provincial Park are complex and varied. Transverse faults, normal faults, thrust faults, and anticlinal and synclinal folds create the complex geological environment (McMechan 1989). The rock types in Peter Lougheed Provincial Park are comprised of various sedimentary rocks. These rocks vary in age from Precambrian to Cretaceous. In the area of Highwood Pass, several north-west south-east trending thrust faults exist, including the Lewis Thrust Fault (McMechan 1989). The topples located on *Rock Glacier Site* are in the Rundle Thrust Sheet (Bielenstein *et al.* 1971). Transverse faults, perpendicular to the thrust faults also complicate the structural geology in Highwood Pass. In fact, there are transverse faults located in gullies in the proximity of *Rock Glacier Site* (McMechan 1989).

The bedrock geology at *Rock Glacier Site* was mapped at a scale of 1:2 500. The mapping of bedrock geology included determining lithology, bedding orientations, thickness of bedding, joint orientations, and spacing of joints. The 1:2 500 scale bedrock geology map produced from the field work is shown in Figure 4.1. The complete stratigraphic profile for *Rock Glacier Site* is shown in Figure 4.2.

Three different rock types were identified during field mapping. A fine grained sandstone, shale, and a quartzite were found to comprise the various lithologies at the site. These results were in conflict with some of the lithologies indicated by the different bedrock geology maps which were referenced to plan the field mapping. Neither the Geological Map of Peter Lougheed Provincial Park (McMechan 1989) or the Bedrock Geology Maps of the Alberta Foothills and Rocky Mountains (Bayrock and Reimchen 1975) indicated the presence of quartzites, while both references indicated siltstones which were not found in the field. However, the bedrock geology map, Geology of Rocky Mountain Foothills and Front Ranges in Kananaskis Country, Southwest of Calgary, Alberta (McMechan 1993), did indicate the presence of quartzites at the top of the slope at *Rock Glacier Site*.

JURASSIC

Fernie Formation

Shales (FSh)	Light grey to brown, closely jointed, and weathered near surface (>200m thick)

TRIASSIC

Spray River Group

Sandstone (Sst)	Light grey with close jointing. Weathers to a dark brown colour (350m thick)
Sandstone/Shale (Sst/SRSh)	Sandstone with interbedded shales (100m thick)
Shale/Sandstone (SRSh/Sst)	Shales with interbedded sandstones (50m thick)
Intact Shale (SRSh)	Light to dark grey, lightly weathered at surface, and closely jointed (15m thick)
Weathered Shale (WSh)	Highly weathered in place. Classified as a geotechnical soil with relict discontinuities (40m thick)

UPPER CARBONIFEROUS and PERMIAN

Rocky Mountain Super Group

Quartzite (Qtz)	Light grey to beige, medium grained, and closely jointed (150m thick)

Figure 4.2: Stratigraphic profile for regional bedrock geology map of *Rock Glacier Site*.

Siltstone, as expected from the bedrock geology maps (McMechan 1989 and Bayrock and Reimchen 1975) was not found to be present, however, this may only be due to a difference in judgment. The sandstone identified was noted as fine grained. It is possible that other geologists might argue that this same material is, in fact, a coarse grained siltstone, where the difference between a fine grained sandstone and a coarse grained siltstone is insignificant from a stability point of view. Since the material's density and internal angle of friction were measured (to be discussed in chapters five and six), the name given to the rock is of no consequence in terms of the stability analysis. Despite this, it is felt that due to the gritty feel and appearance of the rock in question that it was indeed a fine grained sandstone. The Lexicon of Canadian Stratigraphy (Glass 1990) and the work of Irish (1965) supports the presence of a sandstone in the Spray River Group and Hu (1991) also mapped the same rock as being a sandstone. Therefore, the rock will be referred to as a sandstone throughout this report.

The rock at the upper section of the site was found to be a quartzite, from the Rocky Mountain Group, as indicated by McMechan (1993) (Fig. 4.1). This was also in conflict with two of the bedrock geology maps which were consulted (McMechan 1989 and Bayrock and Reimchen 1975). These maps indicated chert and dolomite would be expected at the upper section of the site. Chert nodules and dolomite were present in ample quantities as debris in the gullies, indicating their presence at higher elevations on the slope. However, the quartzite was in excess of 150 m in thickness, extending well beyond the area of interest for this site investigation, resulting in no need to further examine the lithologies of the rocks located up slope from the quartzites. Hu's (1991) research on toppling in Highwood Pass supports the fact that the rock at the upper section of the site was a quartzite. Hu's (1991) research also indicated all toppling movements in the Highwood Pass area to be located solely in the Triassic sandstones and shales of the Spray River Group, indicating no reason to investigate the rocks located up slope from the quartzites.

At the upper plateau of the site, in the Spray River Group, there is a weathered shale, approximately 40 m thick, located down slope from massive quartzite beds and up slope from thin intact shale beds (Fig. 4.1). This weathered shale zone can be seen in profile in Figure 4.3a. Field observations suggested that the shale was completely weathered. There is a classification scheme available which can be used to more systematically describe the degree of weathering of a rock mass. Using the system proposed by Price (1993), a rock mass weathering rating of -10 can be determined for the shale at the upper plateau. A rating of -10 corresponds to a Class D2 material. In descriptive terms, a Class D2 material is a geotechnical soil with relict discontinuities. Price (1993) correlated his proposed classification to the classification scheme used in the British Standards for Site Investigations. The same material, according to the British Standard for Site Investigations, is classified as completely weathered with original structure and texture largely intact (Price 1993). As a result, this zone of material will be referred to as the completely weathered shale zone or simply as the weathered shale zone throughout this report.

The bedrock geology maps referenced above (McMechan 1989 and McMechan 1993) and the work by Hu (1991) all indicate the presence of shales of Jurassic age under the colluvium at the toe of the slope. Although these shales were not identified directly at *Rock Glacier Site*, they were identified in the immediate area. Since they are located at the base of the slope and are buried under a significant amount of colluvium, no further consideration will be given to this rock except that Jurassic shales are illustrated on the bedrock geology map of *Rock Glacier Site* (Fig. 4.1).

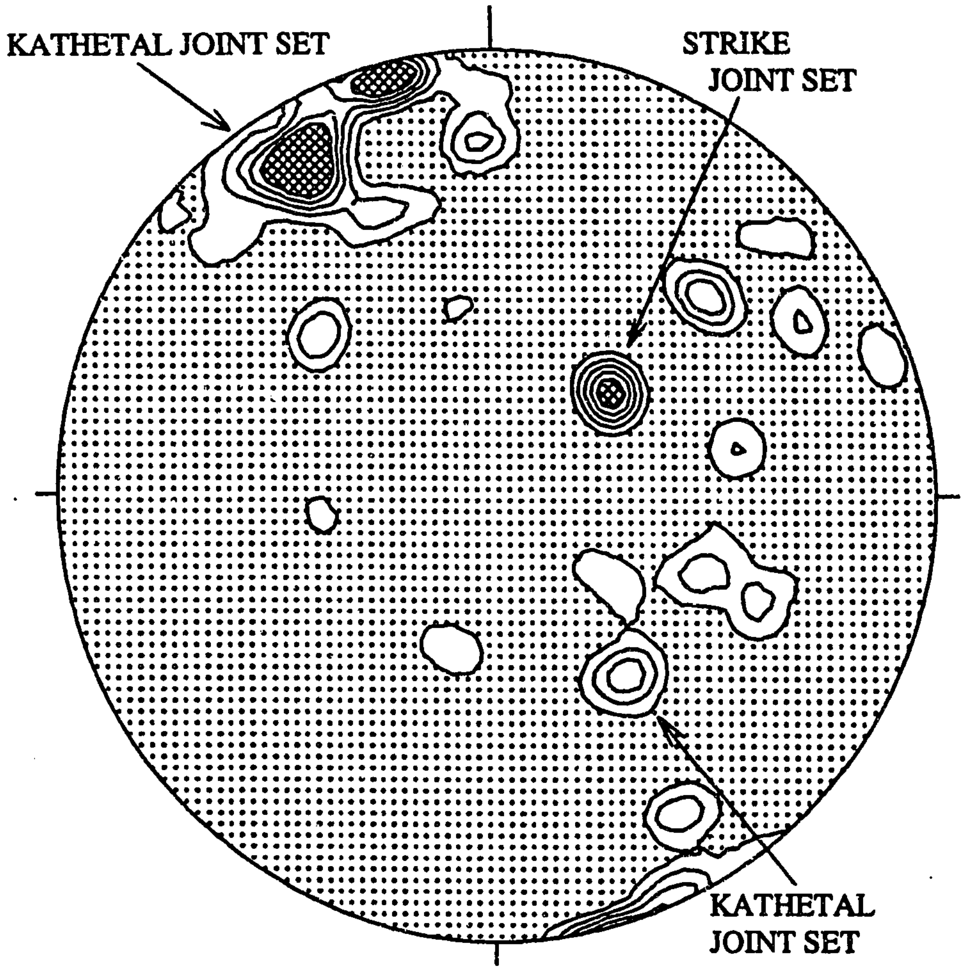
4.4 Bedrock Geology (Structure)

The orientations of the bedding indicated on Figure 4.1 were determined by the use of a contouring method. Many measurements of dip angle and dip direction were recorded during the process of field mapping. Measurements for each area were plotted and contoured individually on stereographic plots in order to determine representative

orientations for that area. The contouring of field data was facilitated using a computer program called FABRIC (Starkey 1977). The center of the maximum concentration of points plotted with the program indicates the orientation which is statistically the most representative. Similarly, the measurements recorded in the field of joint orientations were also contoured using FABRIC (Starkey 1977) to obtain representative orientations for each area. The statistically representative joint and bedding orientations will be discussed in detail in the next few paragraphs.

The rock mass at *Rock Glacier Site* was found to be closely jointed. The close spacing of the joints can be seen in the more massive sandstone beds located deep in the gullies (Fig. 4.3b). The results of contouring the joint orientations for each site individually and the results from contouring the joint orientations together for the entire site both indicated the presence of essentially only three main joint sets. The plotting and contouring of the overall data for all joint measurements (Fig. 4.4) clearly indicates two sets of kathetal joints and one set of strike joints.

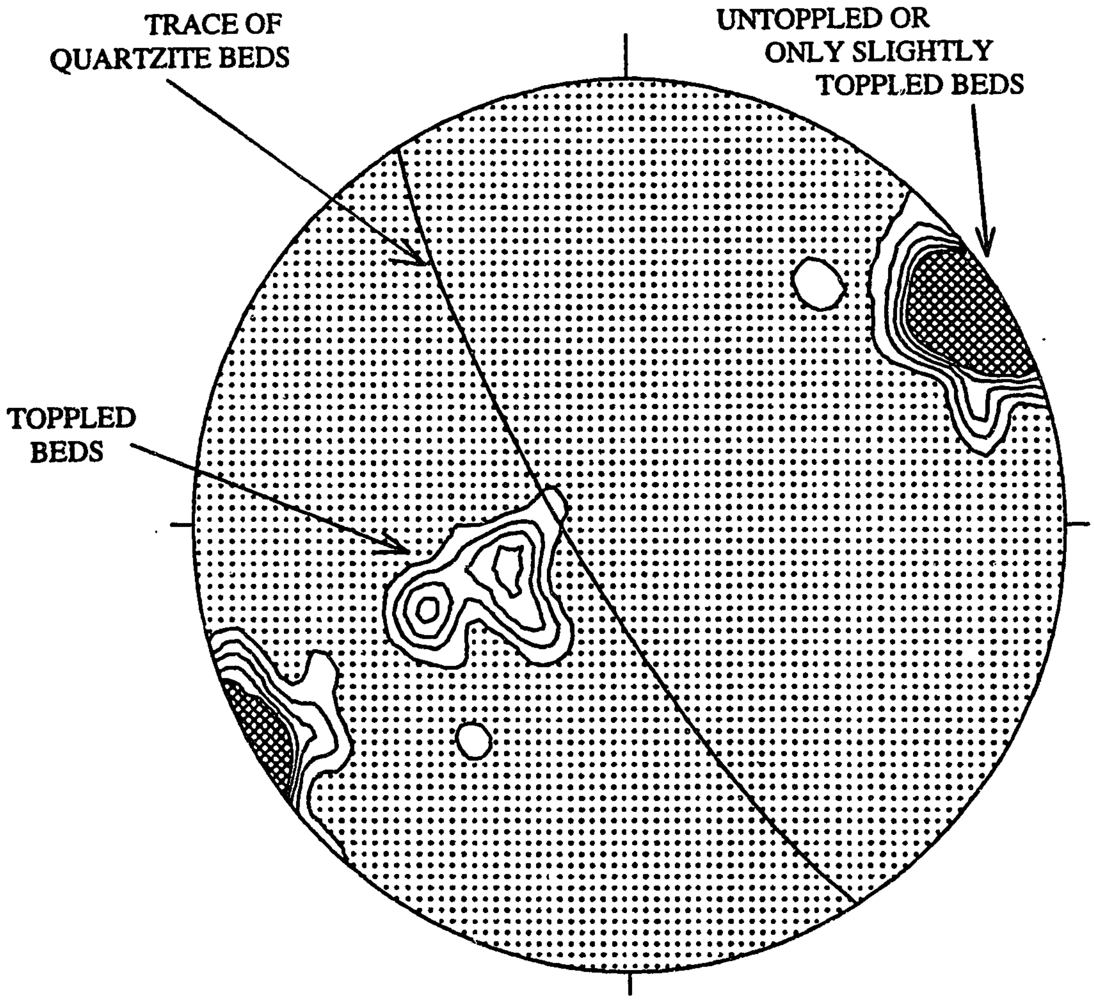
To estimate the overall trends of orientation for bedding with respect to the entire site, all the orientations were plotted and contoured together. The results of contouring the bedding orientations are shown in Figure 4.5. The orientations of bedding can be seen as being divided into three groups. One group of bedding orientations is the regional orientation of untoppled beds. The quartzite beds located near the top of the slope represent the untoppled orientation. These beds have an orientation of 79°~238° (dip and dip direction). The second group of bedding orientations, having an orientation of 86°~239° (dip and dip direction) represents the average orientation of beds which have either not toppled or have toppled only a small amount. The other group of points having an orientation of 35°~071° (dip and dip direction) represents the average orientation of beds which have toppled to a significant degree. From these results, a regional sense of the rotation about strike and dip can be obtained. On a site specific scale, the amount of rotation of the beds varied significantly. On a regional scale, the beds which have toppled have rotated about strike an average of 13 degrees and rotated



Contours 2 4 6 8 10

Sample Size = 235
Lower Hemisphere Input Data
Lower Hemisphere Plot

Figure 4.4: Stereographic plot of all joints measured at *Rock Glacier Site* (Lambert equal area projection).



Contours 2 4 6 8 10

Sample Size = 248
Lower Hemisphere Input Data
Lower Hemisphere Plot

Figure 4.5: Stereographic plot of all bedding plane poles measured at *Rock Glacier Site* (Lambert equal area projection).

with respect to the dip an average of 66 degrees. Some topples rotated about strike only one or two degrees while some were measured as having rotated 24 degrees. The amount of rotation with respect to dip varied from only one or two degrees to rotations as large as 88 degrees. Rotation about strike during toppling may be caused by an uneven distribution of driving forces or it may be a result of an anisotropic rock mass. As a rock mass topples, the rotation about strike and dip are likely to be occurring simultaneously. This suggests that the axis of rotation for each topple is probably unique and complex.

Three different modes of toppling observed at *Rock Glacier Site* were block flexure topples, block topples, and multiple block topples. A large topple at *Rock Glacier Site* was identified by Hu (1991) as a block flexure topple. This topple is located near the toe of the slope in the north gully and is labeled as "A" in Figure 3.2. Block flexure topples are characterized by a gradual change in bedding orientations within the toppling mass (Hu 1991). Upon re-examining this topple, it was discovered that it is in fact a block topple and not a block flexure topple. At the base of the topple, an angular rotation of 58 degrees was measured between adjacent blocks. Block topples are characterized by abrupt changes in orientation between blocks within the toppling rock mass. An angle of 10 degrees or greater between adjacent blocks has been proposed as a guideline indicating an abrupt change in orientation (Cruden and Hu 1994). It is suspected that Hu (1991) wrongly classified this topple because the base of the topple was still covered in snow when he investigated this area. Hu (1991) identified two other topples at *Rock Glacier Site* which he classified as multiple block topples. Both of these multiple block topples are located near the top of the slope. The first of the multiple block topples is located in the center of the slope, marked as "B", while the second is located on the south side of the south gully marked as "C" in Figure 3.2. Multiple block topples are characterized by more than one distinct zone of abrupt change in bedding orientation between blocks within a toppling mass (Hu 1991). These three topples were identified by Hu (1991) during his research on topples on underdip cataclinal slopes in the Highwood Pass area. A more detailed discussion of the stability of these topples and the stability of the entire slope is discussed in chapter seven.

Based on information gathered from the 16 topples studied by Hu (1991), a criterion for predicting toppling style based on the h/b ratio was developed (Cruden and Hu 1994). This criterion stated that block flexure topples occur where the h/b ratios are less than two, while block topples and multiple block topples form where the h/b ratios are larger than two. A total of 20 topples, including three studied by Hu (1991), were identified at *Rock Glacier Site* as part of this study. These 20 topples are listed in Table 4.1 along with the style of toppling, thickness of bedding, and the h/b ratio for each topple. More detailed information regarding each of these 20 topples and their location on the slope can be found in Table 7.1. The question marks after the block flexure topples in Table 4.1 are related to the fact that these topples are difficult to recognize when there is very little angular rotation within the entire rock mass. In order to evaluate the criterion put forth by Cruden and Hu (1994), a plot of toppling style versus h/b ratio of each of the 20 topples examined is shown in Figure 4.6. The results indicate that block flexure topples occur with h/b ratios ranging from 0.1 to 10. The results also indicate that block topples occur with h/b ratios below 1.5 and multiple block topples occurring with h/b ratios ranging from 1 to 6.7. Clearly, these results suggest that Cruden and Hu's (1994) criterion needs to be re-examined. The variations in results can be attributed to the small databases used for forming these empirical relationships. This problem is discussed further in chapter 10.

There are also inherent problems with using the h/b ratio for establishing any criteria. Although the bedding thickness and joint spacing measured by Hu (1991), for the three topples located on *Rock Glacier Site,* are close to the values measured for this research, they differ enough to give significantly different h/b ratios. This indicates that h/b ratios may be very sensitive to variations in mapping style. Another point to consider is that the h/b ratio of a rock mass may change as the mass topples. Specifically, the spacing between joints may decrease as the topple progresses by the formation of new joints.

Topple Identification	Bedding Thickness (cm)	h/b Ratio	Type of Topple
A	100	0.2	Block Topple
A	150	0.1	Block Topple
B	15	1.7	Multiple Block Topple
C	1.5	3.3	Multiple Block Topple
C	1.5	3.3	Multiple Block Topple
D	30	0.7	Block Topple
E	1.5	6.7	Multiple Block Topple
F	10	1.5	Block Topple
G	5	1	Multiple Block Topple
H	5	2	Multiple Block Topple
I	5	2	Multiple Block Topple
J	20	1.5	Multiple Block Topple
K	4	2.5	Multiple Block Topple
L	65	0.4	Block Topple
M	200	0.07	Block Flexure ?
N	100	0.2	Block Flexure ?
O	1.5	10	Block Flexure ?
P	1.5	10	Block Flexure ?
Q	1.5	*N/A	Block Topple
R	11	*N/A	Multiple Block Topple

* NOTE: h/b ratio not available.

Table 4.1: Toppling styles, thicknesses, and h/b ratios for the 20 topples identified at *Rock Glacier Site*.

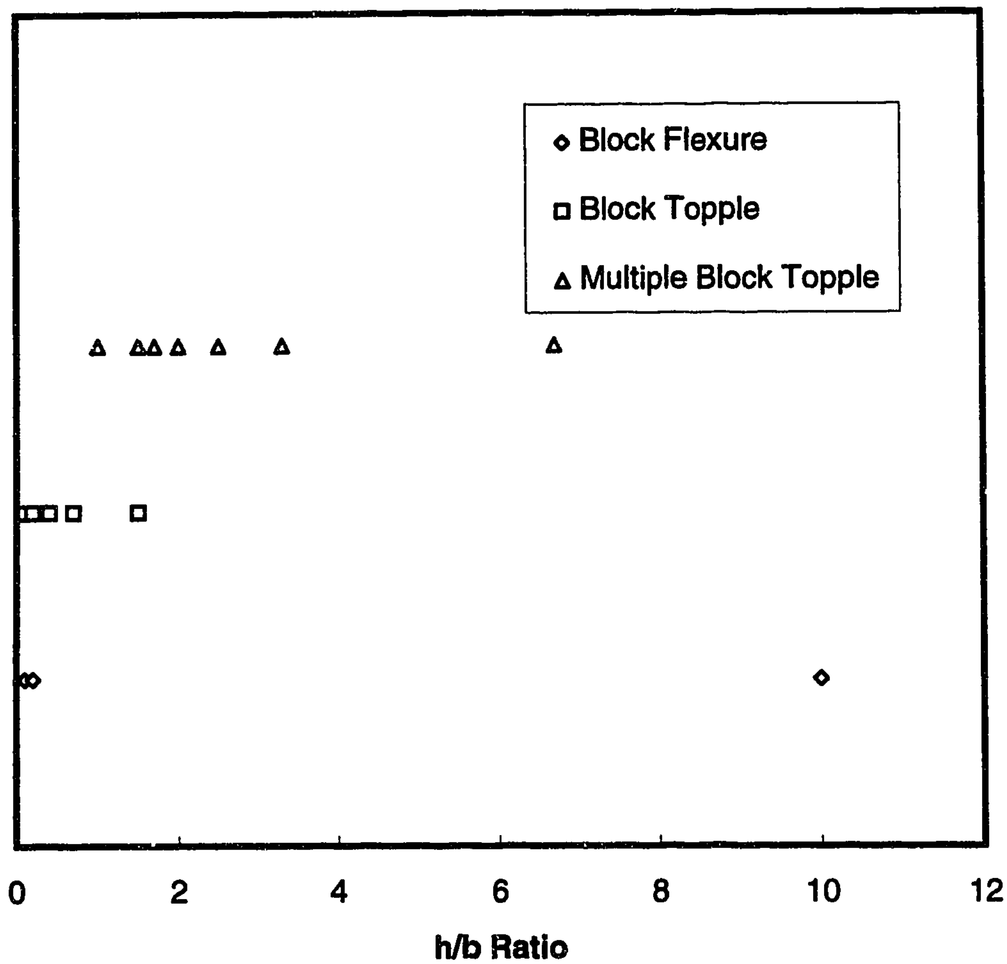


Figure 4.6: Plot of h/b ratios versus toppling style for the topples identified at *Rock Glacier Site*.

This suggests that the h/b ratio measured for a rock mass may be different then the h/b ratio present at the time a rupture might have formed.

Associated with many of these topples were rupture surfaces. Cruden and Hu (1994) were the first to describe these rupture surfaces in any detail. It is believed that rupture surfaces form in response to shear damage in the rock mass during toppling. As a rock mass topples, adjacent beds will experience flexural slip which can cause the rock mass to break apart. Several rupture surfaces were identified while mapping the topples at *Rock Glacier Site*. A large rupture surface, possibly extending under the entire site, was observed over several hundred meters at the base of the north gully. The significance of these rupture surfaces is that they allow the toppling process to continue by destroying obstructions in the mass to the toppling process. Rupture surfaces also provide a surface upon which the toppling mass may simply slide (Cruden *et al.* 1993). A more detailed discussion of rupture surfaces and their role in toppling and the overall stability of the slope is in chapter seven.

4.5 Surficial Geology

Surficial geology maps (Bayrock and Reimchen 1975) indicated colluvial deposits, alluvial fans, and aprons in Highwood Pass. All three of these features were observed either at the site or in the immediate vicinity.

Loose and incoherent deposits, colluvium or talus, were abundant over most of the slope, being a direct result of toppling events and rock falls on the slope. Outspread, gently sloping alluvium, alluvial fans, could be seen at the base of the gullies on either side of the slope (Fig. 3.2). These alluvial fans are possibly a result of snow avalanches, slush avalanches, or debris flows as well as the heavy spring runoff being channeled through the narrow gullies. At the toe of the slope there is a blanket like deposit of unconsolidated material. This deposit is called an apron. The apron is also a direct result of instabilities on the slope. As material has moved down slope, a result of toppling events and rock falls, some of the material has traveled further than average and has been deposited past the toe of the slope forming the more level blanket-like deposits seen at this site.

The surficial deposits were also accurately identified in the airphoto interpretation (Fig. 3.4). By using the aerial photographs and the section profiles, discussed in the next section, the volume of colluvium could be roughly estimated. The volume, or accumulation, of colluvium amassed at the toe of the slope as a result of toppling and rock falls from up slope, was estimated at approximately two million cubic metres (volume calculations are in the appendix). In places, the colluvium was estimated to be as thick as 55 m. The thickness and steepness of the colluvium, inclined at an angle up to 40 degrees, has led to motion of the colluvium solely under the influence of gravity. Shallow sliding movements were frequently observed to spontaneously occur in the colluvium in the absence of any external forces. The act of traversing the colluvium was adequate to trigger even larger sliding movements. Bulges in the mass can be seen on the aerial photographs (Fig. 3.2) and are labeled on the airphoto interpretation (Fig. 3.4). With no evidence of permanent ice within the mass, as mentioned earlier, the slope cannot be accurately called a "rock glacier". It is likely that the downward motion of the rock mass can be explained as simply a result of the effect of gravity. However, ice and snow may, to some degree, play a role in the downward movement of the colluvium during the winter months.

A large amount of material at this site has toppled. Some material has toppled and remained in place atop a larger toppling mass. Some material has completely broken free and accumulated as colluvium or talus on the upper slope. The larger masses of material, having adequate mass, have had sufficient momentum to reach the lower slope section once toppled. These large masses of material have added to the colluvium amassed at the lower section of the slope. Rock falls occur with great frequency at *Rock Glacier Site*. Field observations noted the occurrence of two to three rock falls per hour. Gardner's (1980) research indicated an average frequency of 0.83 events per hour of observation in the entire Highwood Pass area. The average size of these rocks varied between a few centimetres in diameter to tens of centimetres in diameter. The rock fall shadow angle for *Rock Glacier Site* was measured at 31 degrees. Evans and Hungr (1993) defined the rock fall shadow angle as the angle between the distal limit of the shadow and the apex of the talus slope. They proposed an empirical minimum shadow angle of 27.5 degrees (Evans and Hungr 1993). The shadow angle at *Rock Glacier Site* falls within this empirical limit given by Evans and Hungr (1993). The steep ditch alongside Highway 40 appears to have captured any boulders that would otherwise have likely traveled further. It is likely that without the construction of Highway 40, or its continued maintenance, the shadow angle at *Rock Glacier Site* would be closer to 27.5 degrees. This suggests that small rock falls might play a more substantial role than large slope movement events as a source of the colluvium amassed at the base of the slope, a topic discussed in chapter seven, section 7.4.

Both field mapping and airphoto interpretation identified many large blocks or masses of intact rock which at first appear to be insitu, but actually were not. These large blocks or masses of intact rock have detached from their original location and have moved down slope under the influence of gravity with the colluvium (Fig. 3.2). This fact was verified by correlating the lithology and bedding thickness with insitu rock further up slope.

The areas of active accumulation of debris from topples and rock falls can be easily identified in the field and on the airphoto (Fig. 3.2). Areas of inactive accumulation have a dense cover of lichen. For the most part, the entire lower slope has been inactive for many decades or more, as indicated by the dense lichen cover. The upper slope sections are currently active with debris accumulation and show no signs of lichen growth on the debris. Lichen grows very slowly, and where present indicates an area undisturbed by active accumulation of debris. It is possible to estimate the age of the lichen and thus predict a time period over which the accumulation of debris has been inactive. However, for the purposes of this research it is adequate to simply estimate the time periods of inactivity.

4.6 Topography

Measurements of the topography of the slope are required to perform stability analysis. During the process of field mapping, three profiles of the slope were measured. One profile was taken down the center of the slope and one down each of the two gullies. The results of these profiles were compared to existing topographic maps and then topographic contours at 25 m intervals were superimposed onto a 1:5 000 scale airphoto of *Rock Glacier Site* (Fig. 4.7).

To measure the center profile, a straight line traverse was executed down the center of the slope. The profiles of each of the gullies were measured by traversing the line of deepest erosion down each gully. Selected locations, or survey points, down each proposed profile were located on the aerial photographs and then marked in the field. The bearing and inclination between each of the survey locations was found using a Brunton compass. To increase the accuracy of the measurements, the survey points were located on average no more than 150 to 175 m apart, except when it was not possible and greater distances were then required. The distance between the successive survey points was measured with a range finder. The range finder having been accurately calibrated, was able to give distances to within +- 1%. Once the results were compiled, the profiles were adjusted using photographs taken of the slope in profile. The results of all three profiles are plotted in Figure 4.8.

Using the topographic profile of the slope, the bedrock stratigraphy, and the bedrock structure determined in the previous sections, a geological cross section of the site was created. The geological cross section is illustrated in Figure 4.9.

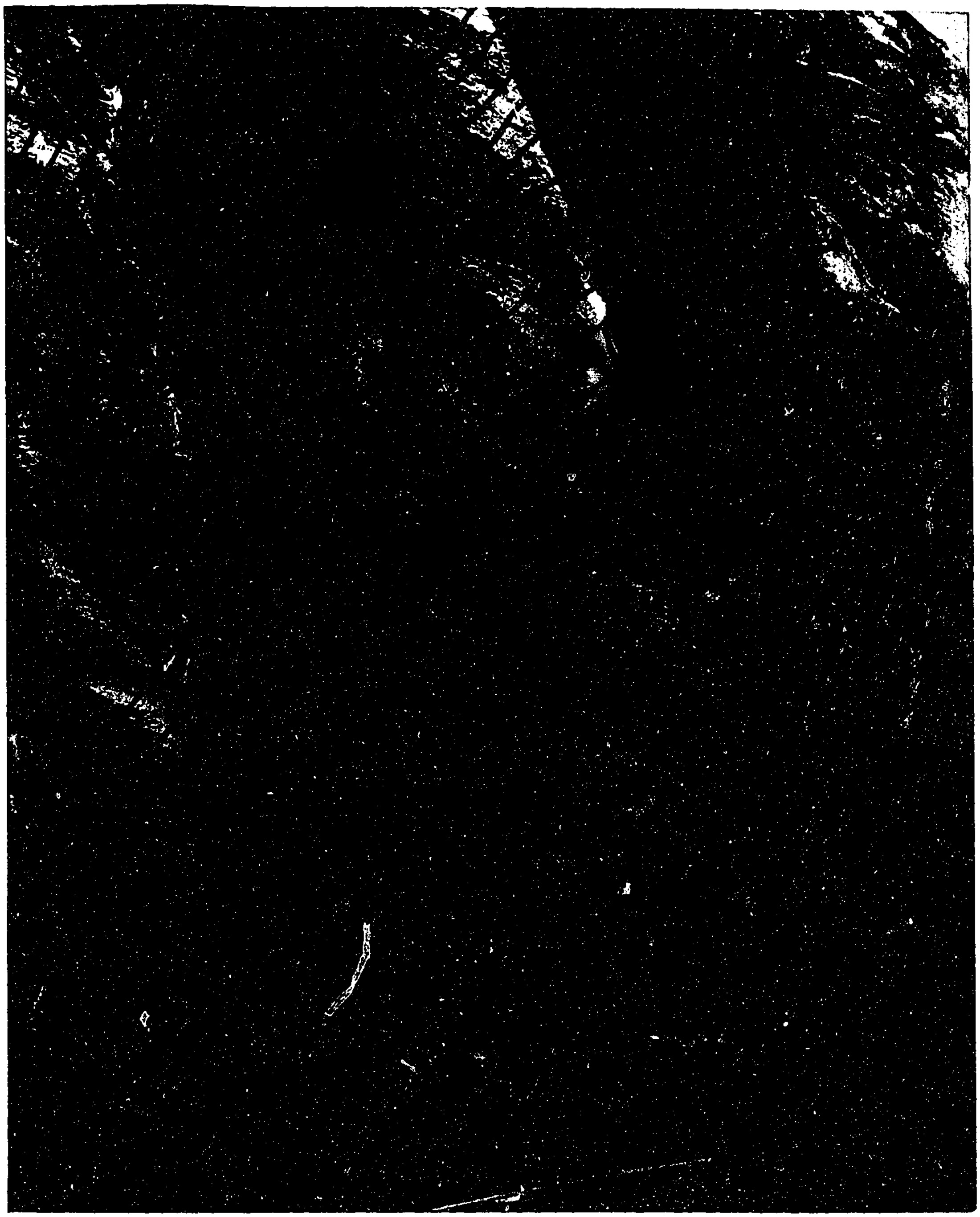


Figure 4.7: Airphoto of *Rock Glacier Site* showing topographical contours every 25 m (1: 5 000 scale). Bearing along highway is 140 degrees.

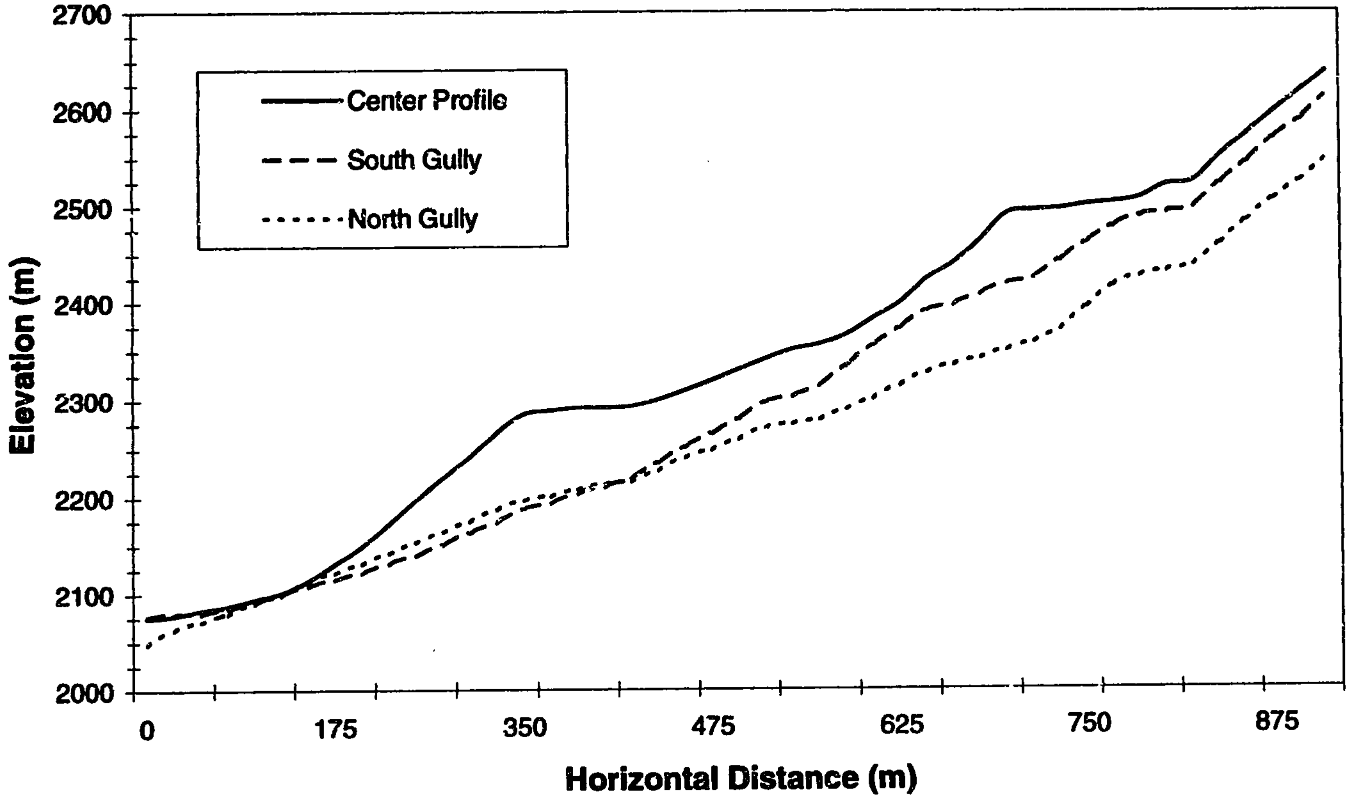


Figure 4.8: Slope profiles for *Rock Glacier Site*.

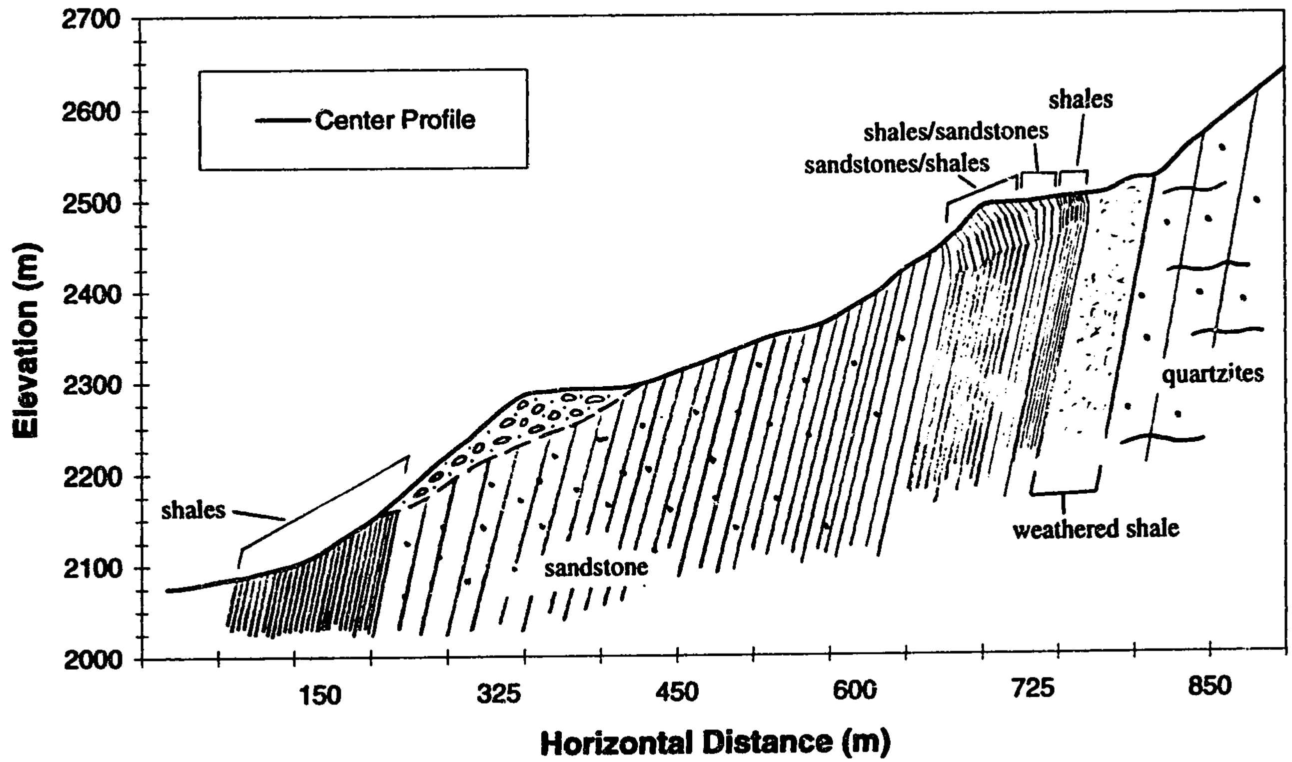


Figure 4.9: Geological cross-section for *Rock Glacier Site*.

To estimate the volume, or accumulation, of displaced material which has amassed at the base of the slope, a pre-movement profile was required. Since the adjacent slopes, located on the same side of the valley, have not moved, are similar in lithology, and similar in orientation, they were considered to be a close approximation to the pre-movement conditions of *Rock Glacier Site*. The slope which was selected to be the most representative of the pre-movement condition was located approximately 750 m to the north of *Rock Glacier Site*. The profile of this slope was superimposed onto the center profile of *Rock Glacier Site* and used as a pre-failure profile for further analysis (Fig. 4.10).

4.7 Conclusions

The field mapping program executed at *Rock Glacier Site* yielded significant amounts of detailed information that can be used to analyze the stability of the slope. A general reconnaissance of the complete area provided a method of verifying the airphoto interpretation. The bedrock geology was completely mapped at a scale of 1:2 500 for the entire site. Mapping the bedrock provided information about the different lithologies, stratigraphic profile, bedding orientations, spacing between bedding planes, joint orientations, and joint spacing. The h/b ratios calculated for the 20 topples identified at *Rock Glacier Site* were compared to the criterion proposed by Cruden and Hu (1994) which related h/b ratios to the style of toppling. The results of this comparison indicate that Cruden and Hu's (1994) criterion should be re-examined. The remaining data was compiled using a software program which allowed statistically representative bedding and joint orientations to be determined. Regional trends of bedrock orientation, toppled rock masses, and joint sets were also obtained with the computer software. The surficial geology was mapped in detail and the zones of active and inactive debris accumulation were identified. Topographical profiles were measured down the center of the slope and down each of the bordering gullies. A pre-movement profile was estimated using the slopes adjacent to *Rock Glacier Site* which

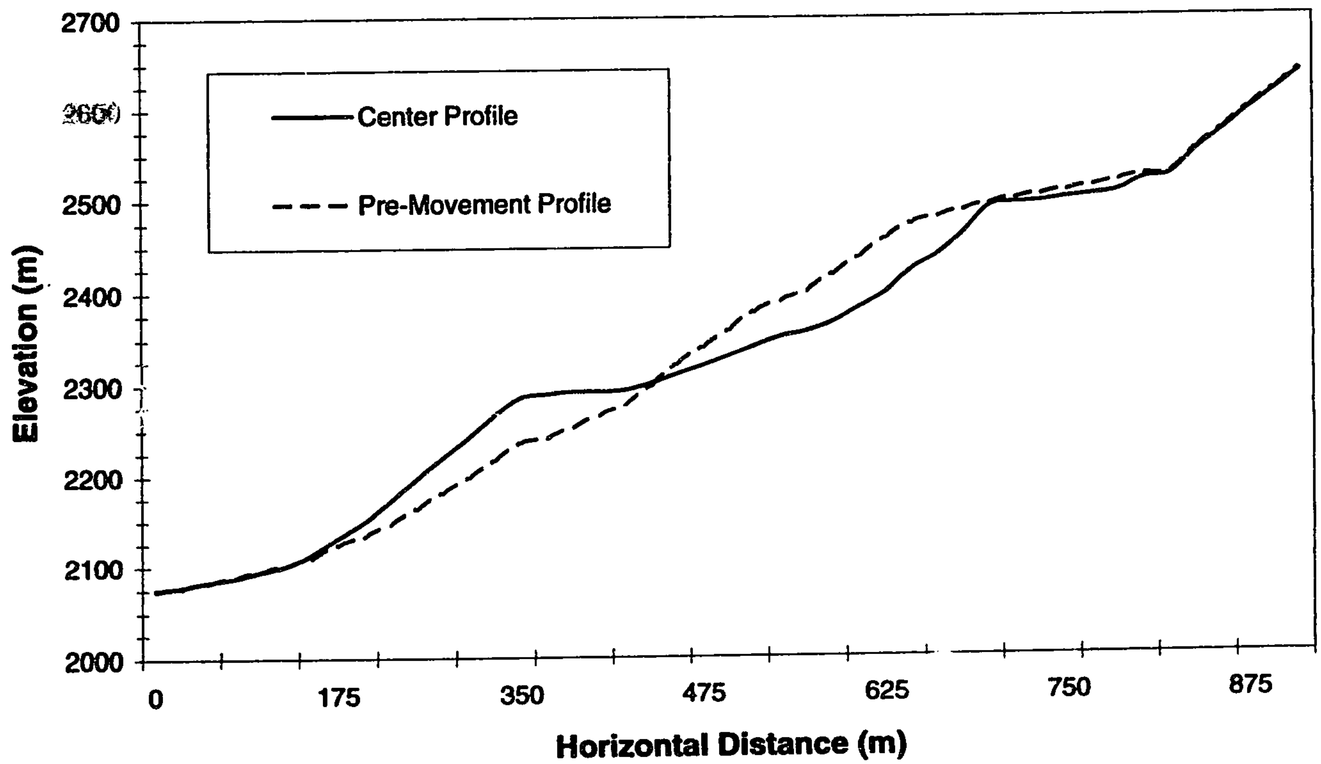


Figure 4.10: Center profile and pre-movement profile at *Rock Glacier Site*.

have not moved. The approximate volume of colluvium at the toe of the slope was calculated using the topographical profiles and the surficial deposits were identified.

CHAPTER 5

INSITU FIELD TESTING

5.1 Introduction

The reasons for using insitu field testing during the course of this project were both essential and complementary. The density of weathered material could only be measured using insitu density tests, while the friction angles and rock hardness of the sandstone and shale could have been measured in the laboratory but instead were measured directly in the field.

The results from field mapping indicated that in a regional sense, all the material down slope from the competent quartzite is undergoing toppling to some degree. Upon close examination, a zone, approximately 40 m thick, of completely weathered shale separated the competent quartzite from the toppling shale and sandstone beds (Fig. 4.1). This highly weathered shale appeared to have been altered, either as a result of the toppling process, or by another process which may have been the mechanism which initiated the topple. To determine the amount of expansion which the shale zone had undergone, it was essential to measure the insitu density of the weathered shale. The testing method selected to determine the insitu density of the weathered shale directly in the field was the rubber balloon method (ASTM 1993a).

The complementary aspect of the insitu field testing method relates to measuring the friction angles and rock hardness. The friction angles of the toppling material are required in order to analyze the stability of the slope. Knowing the rock hardness, or uniaxial compressive strength, of the different rocks present at the site is a good indicator whether slope failure through the rock mass is likely to occur (Herget 1977). Traditionally, samples would be collected in the field, transported to the laboratory, and appropriate tests performed to determine their angle of friction and their uniaxial compressive strength. Since transporting samples carefully over long distances is cumbersome and laboratory testing can be time consuming, a different approach was taken in this project. Insitu tests using a portable tilting table were used to determine the friction angles directly in the field. The portable tilting table method has been shown to give good results and proved to be faster and simpler than more traditional laboratory tests (Hu 1991). The uniaxial compressive strength of the different rocks found at the site was estimated directly in the field by striking with a rock hammer and noting the response (Herget 1977). The three different insitu tests used for this project are described in detail below.

5.2 Friction Angles

In order to evaluate the stability of the slope, the friction angles of the discontinuities in the rock mass must be determined. Using a portable tilt table developed at the University of Alberta (Hu and Cruden 1992b), friction angles were determined directly in the field. Research has shown that the friction angles found using the portable tilt table are close to those in standard laboratory tests (Hu and Cruden 1992b).

Two series of tests to determine the friction angles of the discontinuities were performed directly on the slope. For these tests, flat and relatively smooth samples of sandstone were selected from the upper talus slope. There are two reasons for testing only the sandstone and not the shale. First, the sandstone, being more competent than the shale will be more likely to control the overall stability of the slope due to its greater strength and more favorable position, being down slope from the shale beds. Bromhead (1992) described shales as less competent than sandstones, stating that slope failures are likely to be seated in the shale strata. However, when vertically interbedded shale and sandstone beds exist, as at this site, the sandstone beds contribute more to the strength of the rock mass. Secondly, relatively smooth and flat pieces of shale could not be found in large pieces to test on the portable tilt table. Samples were collected from the talus located immediately below the toppling mass near the top of the slope. The approximate size of samples used was 3 x 10 x 10 cm. The first series of 10 tests were completed with sandstone samples being unaltered and the second series of 10 tests completed where the sandstone samples had been ground together for approximately five minutes. The samples used for the second series of tests were the same samples as were used in the first series of tests. The two blocks were placed onto the portable tilt table one on top of the other. The bottom block was butted up against a support affixed to the table while the top block was free to move. The table was then allowed to tilt at a rate of 8 to 10 degrees per minute. As soon as the top block had slid 2 mm, the rotation of the tilt table was stopped and the angle of inclination of the table was measured using a Brunton compass. A detailed description of how to use the portable tilt table is given by Hu (1991). The results found using the portable tilting table indicated that the undisturbed samples had an angle of friction of 39 degrees while the samples which had been ground down had an angle of friction of 37 degrees. The results indicate that the friction is reduced slightly when the samples are subjected to wear along their surfaces. The results from all 20 tests are compiled in Table 5.1.

Since Hu (1991) also tested samples of sandstone from the Highwood Pass area it would be prudent to compare the results. Hu (1991) performed tests on samples from various areas in Highwood Pass. However, all of the samples were taken from the Triassic Spray River Group which means the results from the two studies can be correlated. Hu (1991) found the angle of friction for sandstone to vary from 28 to 36 degrees. Although the results found in this study are not within the range given by Hu (1991), they are however, close to the largest angle of friction found by Hu (1991).

5.3 Insitu Density Tests

The shale located at the upper plateau was highly weathered in place (Fig. 4.1). The weathering resulted in the shale appearing to have undergone expansion. To determine the extent of the weathering and expansion, the insitu density of the shale needed to be measured. Once the insitu density was found, then it could be compared to the

Weathered Sandstone Samples

Sample No.	Friction Angle (degrees) unaltered samples	Friction Angle (degrees) ground samples
1	39	36
2	41	37
3	39	37
4	39	37
5	39	36
6	39	37
7	40	37
8	39	36.5
9	39	37
10	40	36

Table 5.1: Summary of results from Portable Tilting Table.

unweathered density of the same material and the degree of expansion due to weathering could be determined.

The method chosen to perform the insitu density tests on the weathered shale was the rubber balloon method (ASTM 1993a). This method was chosen for this site for a number of reasons. The accuracy of the method is considered to be relatively high and the consistency of the results acceptable. The rubber membrane which is pressurized with water is a very effective method to measure the volume of a small excavated hole. The size and weight of the apparatus made it feasible to transport to the site, which is located at an elevation of approximately 2550 m. The apparatus weighed approximately four kilograms, was under one metre in length, and required two litres of water to operate.

Insitu density tests were performed with the rubber balloon method at three locations on the weathered shale (Fig. 3.2). ASTM standard D2167-84 (ASTM 1993a) was followed when performing the tests on the weathered shale. Four tests were performed at each of the three sites. The three sites were located: at the top of the slope, on the north facing side of the slope, and on the south facing side of the slope. Using a small shovel, a suitable site was prepared at each location. The shovel was used to remove the material close to the surface and provide a level area to perform the tests. Excavating was difficult on the top and south side of the slope, thus the area was only excavated to a depth of 0.5 - 0.75 m at these two locations. Excavating was much easier on the north slope. However, the material up slope persistently failed into the area being excavated. Thus, only a depth of approximately 1 m was excavated. Once the area was prepared, a small amount of weathered shale was excavated and the rubber balloon apparatus was used to measure the volume of material removed. The material removed was bagged and labeled to be taken back to the laboratory to be weighed and used in other laboratory tests.

The insitu densities found using the rubber balloon method from each location are compiled in Table 5.2. The results showed the insitu density to be lower on the north side of the slope when compared to the other two locations. These results were not unexpected because the north side of the slope appeared to be more weathered at the surface than the other two locations. During the summer field season, the north side of the slope was observed to be nearly always in the shade. Due to this, the area stayed moist for longer periods of time after precipitation and would stay snow-covered for longer periods of time compared to the south side and top center of the slope. The average water contents measured for each of the three locations were 14.6%, 14.9%, and 11.5% for the top location, the north side, and the south side of the slope respectively.

A second insitu density testing procedure was also used to measure the insitu density of the shale. At each of the three locations where the rubber balloon method was used, two Shelby tubes were driven into the ground using a small bearing plate and a one pound rock hammer. The tubes were then carefully excavated, the ends trimmed and sealed, and then transported back to the laboratory for weighing and measuring. The tubes were driven with moderate effort required at the top site and south site. However, the tubes were easily pushed in by hand at the north site.

At each of the sites, and especially the north site, the samples were compressed slightly as the tubes were driven into the ground, presumably due to friction forces between the inner surface of the Shelby tube and the sample being collected. The overall result of this was that the Shelby tubes needed to be driven further into the ground in order to fill the tubes completely. The implications of this were that more weathered shale was required to fill each of the Shelby tubes than the amount expected under ideal conditions. Due to this, the densities calculated using the Shelby tubes were expected to be higher than those found with the rubber balloon method. To compensate for this problem, the samples would need to be extruded and the bottom portion, the portion compressed the most, removed before being weighed and measured, however, this was not possible. Even while exercising extreme care, the samples broke,

Results Using Rubber Balloon Apparatus

	Test No.	Insitu Density (Mg/m^3)	Water Content (%)	Dry Density (Mg/m^3)
Top Center *(location 1)	1	1.48	15.1	1.29
	2	1.65	14.6	1.44
	3	1.70	14.3	1.49
	4	1.48	14.2	1.29
North Slope *(location 2)	5	1.40	16.0	1.21
	6	1.41	13.4	1.24
	7	1.39	15.3	1.20
	8	1.29	15.0	1.12
South Slope *(location 3)	9	1.65	11.5	1.48
	10	1.53	12.0	1.37
	11	1.68	12.4	1.50
	12	1.40	9.9	1.28

Results Using Shelby Tubes

	Test No.	Insitu Density (Mg/m^3)	Water Content (%)	Dry Density (Mg/m^3)
Top Center	A	2.04	12.7	1.81
	B	1.92	14.3	1.68
North Slope	C	1.77	13.8	1.56
	D	1.73	15.0	1.50
South Slope	E	2.06	12.3	1.84
	F	1.82	16.5	1.56

***Note: Indicates location of test site on Figure 3.2.**

Table 5.2: Insitu densities at the three test locations at the upper plateau (weathered shale zone).

separated, and crumbled while being extruded from the tubes. Thus, the only values of insitu density found using the Shelby tubes were based on the entire length and were consistently higher than the results found using the rubber balloon method.

The insitu densities from the three sites calculated using the Shelby tubes are also compiled in Table 5.2. Since the results were expected to be consistently higher and proved to be so, the results found with the Shelby tubes were not used for any calculations, instead only the results from the rubber balloon method were used since they were considered to be more accurate.

5.4 Rock Hardness

Estimates of the rock hardness of the different lithologies identified in the field were based on the criteria outlined in the Pit Slope Manual, Chapter 2 - Structural Geology (Herget 1977). The uniaxial compressive strengths were estimated with this method based on the response of striking the rock with a one pound hammer. The sandstone required, on average, more than one blow with a rock hammer to fracture. From this, the uniaxial compressive strength of the sandstone was estimated at approximately 80 MPa. The intact shale required only one blow with a rock hammer to fracture, indicating a uniaxial compressive strength of approximately 25 MPa. The completely weathered shale was difficult to cut with a hand spade and molded with great difficulty. This indicated that the completely weathered shale was classified as soil with an approximate uniaxial compressive strength of 0.15 MPa. The quartzite required many blows with a rock hammer to fracture. From this, the uniaxial compressive strength of the quartzite was estimated at approximately 200 MPa. Although these results are approximate, they do indicate the relative difference in strength of the different rocks identified at the site. This method of estimating rock hardness is a simpler but less precise method than the Schmidt hammer which can also be directly used in the field (Hoek and Bray 1981).

5.5 Conclusions

The overall results from the insitu field tests are summarized in Table 5.3. The insitu densities found using the Shelby tubes are not included in the table because they were deemed inaccurate and were not used for any calculations.

The portable tilting table was successfully used to determine the friction angles of the samples of sandstone tested in the field. A friction angle of 39 degrees was measured for the unaltered samples of sandstone. A friction angle of 37 degrees was measured for samples which had been ground together prior to testing. The grinding process destroyed many asperities on the surface of the samples giving rise to the lower friction angles which were measured.

The rubber balloon method proved to give consistent results when measuring the insitu density of the weathered shale zone. Observations made in the field that the insitu density of the weathered shale zone at the surface would be lower on the north side of the slope were confirmed with field testing. The densities found with the Shelby tubes were consistently higher than the densities found with the rubber balloon method. The higher results were expected based on observations in the field that the samples in the Shelby tubes were being compressed while being driven into the ground.

The rock hardness, or uniaxial compressive strength, of the different rocks identified at *Rock Glacier Site* were approximated by observing the response of being struck with a rock hammer. The results of rock hardness found in the field are only approximate, however, they provide a good estimate of the relative strengths between the different rocks tested.

Friction Angles:

Average Friction Angle of Weathered Sandstone:	39 +/- 0.7 degrees
Average Friction Angle of Ground Sandstone:	37 +/- 0.5 degrees

Insitu Densities:

Average Insitu Density of Weathered Shale (located at upper plateau):

Top Center:	1.58 Mg/m^3
North Slope:	1.37 Mg/m^3
South Slope:	1.57 Mg/m^3

Average Bulk Unit Weight of Weathered Shale (located at upper plateau):

Top Center:	15.5 kN/m^3
North Slope:	13.5 kN/m^3
South Slope:	15.4 kN/m^3

Rock Hardness:

Estimated Uniaxial Compressive Strengths:

Weathered Shale:	0.15 MPa
Intact Shale:	25 MPa
Sandstone:	80 MPa
Quartzite:	200 MPa

Table 5.3: Summary of results from insitu field testing.

CHAPTER 6

LABORATORY TESTING

6.1 Introduction

The following chapter outlines all laboratory testing related to the Kananaskis research project. The majority of the testing focuses on the weathered Triassic shale from the upper plateau located at an elevation of 2550 m (Fig. 4.1). The laboratory tests include: density tests, grain size analysis, Atterberg Limits, swelling tests, slaking durability tests, calcium carbonate content tests, and frost susceptibility tests.

Interest in the weathered shale relates to it being a possible mechanism for toppling of the shale and sandstone beds located down slope from it. Above the weathered shale zone there are competent quartzite beds and directly below it there are shale and sandstone beds which are toppling (Fig. 4.1). The dip of the bedding changes abruptly from the quartzite to the toppled shale and sandstone beds on the down slope side of the weathered shale. Upon exposure to weathering, the shale appears to have expanded, either by mechanical means or chemical processes causing the shale and sandstone beds to topple down slope. The action of frost and ice lens formation in the weathered shale might have caused the toppling process. To explore this last possibility, the laboratory tests listed above were used to determine whether the weathered shale is indeed susceptible to frost and ice lens formations.

6.2 Density Tests

To supplement the insitu density tests, and to provide the necessary information for the stability calculations, the density of the sandstone, quartzite, and intact shale were measured in the laboratory. The unit weight (density) of the sandstone at its natural field water content was found to be 27.4 kN/m^3 (2.79 Mg/m^3). The unit weight (density) of the quartzite at its natural field water content was found to be 27.1 kN/m^3 (2.77 Mg/m^3). The unit weight (density) of the intact shale at its natural field water content was found to be 22.8 kN/m^3 (2.33 Mg/m^3).

In an attempt to estimate the amount of expansion in the weathered shale at the upper plateau, density tests were performed on fragments of shale retrieved from the site which were believed to be unweathered. Unweathered fragments were tested from the top of the slope, the north face of the slope, and the south face of the slope (Fig. 3.2). The fragments were retrieved during the process of excavating areas to perform the insitu density tests with the rubber balloon test method. The densities found for the three locations were as follows: unweathered fragments from the top of the slope had a unit weight (density) of 24.0 kN/m^3 (2.45 Mg/m^3), unweathered fragments from the north face had a unit weight (density) of 24.1 kN/m^3 (2.46 Mg/m^3), unweathered fragments from the south face had a unit weight (density) of 23.6 kN/m^3 (2.41 Mg/m^3). The natural water content in the field for the unweathered fragments from the center of the slope, the north face, and the south face were 5.86%, 7.75%, and 5.35% respectively. Comparing these results with the insitu bulk unit weights found from the field testing shows that there is significant expansion of the shale due to the material breaking up either from mechanical or chemical weathering. The degree of expansion for the three locations is 54.8%, 78.5%, and 53.2% for the center of slope, the north face, and the south face of the slope respectively. The degree of expansion was calculated by dividing the unit weight of the unweathered fragments by the unit weight of the weathered shale as measured in place.

The much higher degree of expansion on the north slope was not unexpected. Observations made in the field noted that the north slope, while being sheltered from the sun, retained moisture and snow cover for much longer periods of time than the top center or south side of the slope. This leads to more extensive mechanical weathering occurring on the north facing slope. A greater degree of weathering on the north face of the slope, at least at shallow depths, is also supported by the lower insitu density recorded on the north side of the slope compared to the insitu densities measured at the other two locations.

6.3 Grain Size Analysis

A grain size analysis was performed on the weathered Triassic shale from the upper plateau. Once pulverized with a mortar and pestle, 100% of the material passed the #200 sieve. To obtain the grain size distribution, standard hydrometer tests were performed on material from the top of the slope, the north side of the slope, and the south side of the slope (Fig. 3.2). The hydrometer tests were performed as described in ASTM procedures D421-85 (ASTM 1993b) and D422 (ASTM 1990a). The grain size distributions determined with the tests indicate that the clay content ranged between 15.6% and 18.5% with an average of approximately 16.9%. This indicates that the balance of the weathered shale, 83.1%, is silt sized particles. Furthermore, the material appears fairly well graded over the range of fines measured. This is supported by the high values of the coefficient of uniformity calculated for each grain size curve and the coefficients of curvature being between 1 and 3 from each of the grain size curves.

6.4 Atterberg Limits

Following ASTM D4318 (ASTM 1990b) standards, the liquid and plastic limits of the weathered shale were found. The tests were performed on samples taken from the top center of the slope, the north side of the slope, and the south side of the slope (Fig. 3.2). The range of liquid limits found were between 25.2% and 26.6% with an average liquid limit of 25.8%. The range of plastic limits found were between 19.0% and 20.4% with an average plastic limit of 19.7%. According to Mitchell (1976), both the liquid and plastic limits found for the weathered shale are low. Using these results, the activity of the shale was determined to vary between 0.34 and 0.39 with an average value of 0.36. The fact that the activity of the weathered shale is low indicates that the material would not be expected to be susceptible to swelling (Bowles 1986). If true, then the expansion of the weathered shale zone would be completely controlled by mechanical processes, possibly ice lens formation or frost heaving and not chemical processes. To verify this, swelling tests on the weathered shale are required and will be discussed in the following section.

6.5 Swelling Tests

The swelling test was performed as a reverse consolidation test. A nominal seating load of 0.25 kPa was placed onto the sample in the odometer and immersed in water for 72 hours. During this time there was no swelling recorded. These results confirmed those found earlier with respect to the Atterberg Limits and the activity of the weathered shale. The low activity indicated that the shale would not be susceptible to swelling.

6.6 Slaking Tests

A Slake durability index test was performed on unweathered fragments of shale retrieved from the weathered shale zone. The test was performed in accordance with ASTM standard D4644 (ASTM 1990c). However, the largest pieces of unweathered shale recovered from the site were not as large as was required by the ASTM standard (ASTM 1990c). Since larger pieces did not exist at the depth excavated into the weathered shale zone, the largest sized fragments retrieved were tested. The fragments tested ranged in size from 12 to 40 g in mass. To further deviate from the ASTM standard (ASTM 1990c), not enough fragments of even this smaller size were available for the test. Only 354.8 g of unweathered shale was available, while the ASTM standard (ASTM 1990c) required between 450 to 550 g. Despite these problems, the slake durability index test was performed with the only material available.

The slake durability index refers to the percentage of material remaining after two cycles of drying and wetting, where the wetting cycles are accomplished by tumbling in a standard sized steel drum for 10 minutes in tap water at 20 degrees Celsius. The results of the slaking test indicated that the unweathered shale fragments, retrieved from the weathered shale zone, had a slake durability index of 84.0%. The material retained in the drum at the end of the test was classified as Type II (ASTM 1990c). Type II implies that the retained pieces consist of both large and small sizes. The material passing through the drum varied from particles which quickly settled to particles which remained in suspension in the water. The slake durability test provides an indication of the degree of mechanical breakdown which is to be expected from alternate wetting and drying cycles. The results of this test indicate that the mechanical breakdown of the shale taken from the zone of weathered shale, as a result of wetting and drying cycles would be low to moderate in action. This result was anticipated due to the low activity of the shale tested. Attewell and Farmer (1976) stated that the slaking tendency of shales increases as the activity increases. Thus, a low activity indicates a low tendency for slaking of shales.

6.7 Calcium Carbonate

When hydrochloric acid was placed on fragments of shale from the upper plateau there was a mild to moderate reaction. When the shale was broken and crushed slightly there was a violent reaction when mixed with hydrochloric acid. In order to establish whether or not the weathered Triassic shale was cemented with calcium carbonate and if so to what degree, the following test was performed. A fragment of air dried unweathered shale was pulverized, weighed, and placed into a 250 ml beaker. 125 ml of 25% hydrochloric acid was then added to the powder in the beaker and allowed to react for approximately 30 minutes. During the first 10 minutes the solution effervesced violently then continued to react moderately for a second 10 minutes. The crushed shale and hydrochloric acid solution was then passed through two stages of filtering. The fines captured by the filters were then dried and weighed. The difference in the weight of the dry fines before and after reacting with the hydrochloric acid represented the amount of calcium carbonate present in the shale as a binding agent.

The results of the test indicated the presence of 8.9% by weight of calcium carbonate in the shale. The presence of calcium carbonate in such a high amount indicates that it is likely acting as a binding agent within the shale's structure. The test outlined above does not follow the ASTM standard for determining the calcium carbonate content of soils (ASTM 1994). This test was not used due to its complexity in procedure and the special equipment required to perform the test. The ASTM standard determines the calcium carbonate content of soils from the measurement of the pressure generated by the evolved carbon dioxide as a result of the reaction with hydrochloric acid (ASTM 1994). A purpose - made closed pressure vessel is required and calibration tests are performed using a range of known masses of pure calcium carbonate (ASTM 1994). The accuracy of the test method which was used to determine the calcium carbonate content of the shale tested was sufficient for the needs of this project. The results only needed to indicate whether the calcium carbonate content was very low or very high, which they did. A high calcium carbonate content suggests that the shale may weaken if the calcium carbonate is removed through dissolution as a result of weathering.

6.8 Frost Susceptibility

There are three basic requirements for frost action to occur in soils; the ground temperatures must be below 0 °C, there must be an adequate supply of capillary or groundwater sufficient to form and supply accumulating ice lenses in the frozen soil, and soil must be susceptible to frost (Phukan 1985). Frost heaving occurs due to the growth of ice crystals into lenses of ice. This growth is fed by the upward flow of moisture pulled by a temperature induced suction gradient to the freezing front (Konrad and Morgenstern 1983). Water can also be supplied by capillary forces to the freezing front from water filled cracks or bedding planes at depth in the rock mass (bedding planes in the weathered shale zone are inclined at approximately 80 degrees).

In the Highwood Pass area today, the temperatures are significantly below freezing for a considerably large part of the year (discussed in section 3.4), even lower temperatures would have been present in the past. The Little Ice Age, ending between 100 to 300 years ago (Fulton 1989) and the period corresponding to the Wisconsin Glaciation some 10 000 to 12 000 years ago (Fulton 1989) would have presented a much colder climate than we see today in the Highwood Pass. Gardner (1980) has even found remnants of Little Ice Age glaciers to still exist on Mt. Rae today. During prolonged periods of sub-zero temperatures, the zone of freezing will extend to greater depths in soil and rock masses (Craig 1990). Thus, longer durations of sub-zero temperatures augment the degree of frost heave associated with ice lens formations. MacKay (1972) supported this when he discussed how ice wedges can form and grow to large sizes at favorable sites around a glacial margin. Peltier (1950) adds that areas adjacent to ice sheets are generally characterized by intense frost action due to the continually low temperatures.

The amount of precipitation in the form of rain and snowfall in the Rocky Mountains provide adequate moisture near the surface and at depth. The mean annual precipitation in Highwood Pass was estimated at 662.1 mm (discussed in section 3.4). Water trickling down slope from melting snow can also provide sufficient moisture to the freezing system. Warm temperatures during the day cause the snow to melt and cold temperatures during the night will result in freezing of these melt waters.

The only question remaining is whether or not the zone of weathered shale is itself susceptible to frost. The completely weathered shale in question has been formally classified as a geotechnical soil with relict discontinuities (Price 1993). Bedding structure still exists, yet the rock crumbles and breaks apart into a soil when disturbed. There are unweathered fragments of shale throughout the weathered mass that display limited strength, yet even these fragments are easily broken down with light mechanical force such that 100% passes the #200 sieve. Due to these characteristics, the zone of completely weathered shale will be treated as a soil for frost susceptibility analysis. However, as a rock, the shale, if susceptible to frost, would heave and support ice lens formations just the same as a soil would.

Traditional frost susceptibility is based on grain size alone. According to Casagrande (1931), considerable ice segregation is expected in non-uniform soils containing more than 3% of particles smaller than 0.02 mm in diameter and in very uniform soils containing more than 10% of particles smaller than 0.02 mm. The weathered shale considered here is uniform and has on average 45% of its particles smaller than 0.02 mm, indicating that the material is frost susceptible. The U.S. Army Corps of Engineers uses a criterion, related to the Unified Soil Classification System, which requires information about grain size distribution and Atterberg Limits (Chamberlain 1981). Using their classification, the weathered shale considered here is designated as medium to extremely frost susceptible, solely on the basis of grain size alone. The grain size distribution is such that there is no need to consider the Atterberg Limits. After comparing laboratory frost heave tests with the two classification schemes above, Chamberlain (1981) rated Casagrande's criterion as 75% reliable and the U.S. Army Corps of Engineers criterion as 91% reliable for predicting frost susceptibility.

A more recent approach for predicting frost susceptibility of soils and the degree of frost heave expected (segregation potential) was based on parameters derived from the specific surface area and the mineral content of the fines fraction of the soil (Davila *et al.* 1992, Davila 1992). Davila, Sego, and Robertson (1993) related these same parameters to the liquid limit of the fines fraction of the soil and the percentage of clay in the fines fraction of the soil. Since the grain size distribution of the weathered shale zone was such that 100% passed the #200 sieve, then the liquid limit determined in the laboratory tests was in fact the liquid limit of the fines fraction of the soil. Similarly, the percentage of clay measured in the hydrometer tests also directly gave the percentage of clay with respect to the fines fraction of the soil. Using the liquid limit of the fines fraction (Llff = 25.8%), the percentage of clay in the fines fraction (CS = 16.9%), and Davila, Sego, and Robertson's (1993) correlation criteria, the weathered shale zone was deemed to be highly frost susceptible and exhibited nearly the maximum segregation potential possible (SPo = 22×10^{-4} mm^2/s°C).

For consideration of frost susceptibility evaluation, the grain size distribution should be based on field grain size distributions. The grain size of the weathered shale was determined in the laboratory. Laboratory samples are first ground and then tested, while in the field the soil is not ground. In this case the results should not differ substantially since only a light mechanical force was required to prepare the sample in the laboratory.

Having proven that the highly weathered shale is extremely frost susceptible, exposed to adequate moisture, and exposed to sufficiently low temperatures, it can be concluded that this material has undergone frost heave and been subjected to ice lens formation. The favorable orientation of the bedding has also contributed to the degree of frost heave and ice lens formation. The weathered shale zone exhibits bedding dipping approximately 80 degrees down slope (Fig. 4.1). The steep bedding allows ample water to infiltrate along bedding planes. During frost heave, the forming ice adheres to the shale and expands perpendicular to the bedding planes. Water held at depth in the bedding planes is drawn to the ice front by suction gradients and capillary forces allowing ice lenses to grow in size. Since expansion is dominantly perpendicular to bedding and the [illegible] slope is stiff and supported by material further up slope, then the expansionary forces push the intact shale and sandstone beds down slope. The depth of frost penetration in the weathered shale is enhanced by the presence of the deeply incised gullies. The freezing front in the weathered shale advances from the top of the slope and from the sides. Once expansion has been achieved, the opening is supported by infilling material (Cruden *et al.* 1993) and by the weathered shale layer relaxing into a continually expanding zone. Successive freeze-thaw cycles over a long period of time cause the intact shale and sandstone beds to experience flexure and begin to topple. At some point, the toppling of the beds causes them to rupture at depth. Several rupture surfaces were found at this site which support this hypothesis. These rupture surfaces will be discussed in detail in chapter seven. Once a rupture surface has formed, the beds may then slide if the friction angle is sufficiently low.

6.9 Conclusions

The various laboratory tests outlined above resulted in significant findings. By knowing the index properties of the completely weathered shale zone, a better understanding of the role it might have played with respect to the toppling process can be determined. The results from all the laboratory tests are summarized in Table 6.1.

Determining the unit weights of the toppled materials is essential in performing a stability analysis on the slope. By comparing the unit weights of the unweathered fragments from the weathered shale zone to the insitu density from the three locations tested, the amount of expansion due to weathering was calculated. The expansion due to weathering at the top of the slope, the north side of the slope, and the south side of the slope was 54.8%, 78.5%, and 53.2% respectively. The higher degree of expansion calculated for the north slope supports the observations made earlier that the north slope was weathered to a higher degree, at least near the surface.

The grain size analysis indicated that 16.9% of the weathered shale zone was comprised of clay sized particles, and that the remaining 83.1% was of silt sized particles. The high percentage of silt indicates that the weathered shale zone is likely susceptible to frost action.

By determining the Atterberg Limits of the weathered shale zone, the activity of the material could be calculated. The liquid and plastic limits of the weathered shale were determined to be 25.8% and 19.7% respectively. Thus, the activity was calculated as 0.36. The low activity of the weathered shale zone indicates the material is not susceptible to swelling. As a confirmation, a swelling test was performed on the material. The test indicted that the unweathered shale does not swell even slightly.

The slake durability index test is a measure of the resistance of the material to weakening and disintegrate from two standard cycles of wetting and drying. The slake

Density Tests:

sandstone	2.79 Mg/m^3	(27.4 kN/m^3)
intact shale	2.33 Mg/m^3	(22.8 kN/m^3)
quartzite	2.77 Mg/m^3	(27.1 kN/m^3)

Calculated Degree of Expansion of Weathered Shale:

Top Center	54.8%
North slope	78.5%
South Slope	53.2%

Grain Size Analysis (weathered shale zone):

16.9% clay with 100% passing #200 sieve.
Cu = 62; Cc = 2.5

Atterberg Limits (weathered shale zone):

Liquid Limit: 25.8%
Plastic Limit: 19.7%

Activity (weathered shale zone):

Activity: 0.36

Swelling Test (weathered shale zone):

Not Susceptible to Swelling.

Slake Durability Test (weathered Shale zone):

Slake Durability Index: 84.0%
Type II material Retained in Drum (both large and small pieces).

Calcium Carbonate Content Test (weathered shale zone):

8.9% by weight Calcium Carbonate.

Frost Susceptibility Test (weathered shale zone):

Highly Frost Susceptible
Segregation Potential: SPo = 22x10^-4 mm^2/soC.

Table 6.1: Summary of all laboratory tests.

durability index of the unweathered fragments of shale retrieved from the weathered shale zone was 84.0%. Lama and Vutukuri (1978) rate a slake durability index of 84.0% as medium, where the range is broken into six categories with medium being third from the lowest rating. The implications of this are that the durability of the unweathered shale to wetting and drying cycles is neither high nor low.

A calcium carbonate content test on the weathered shale indicated that the material was 8.9% by weight calcium carbonate. The presence of calcium carbonate in such a high percentage in the completely weathered shale indicates that it is likely acting as a cementing agent within the shale structure. A high calcium carbonate content suggests that the shale may weaken if the calcium carbonate is removed through dissolution as a result of weathering.

The narrow zone of weathered shale located at the upper plateau of the slope lies between competent quartzite beds and toppled shale and sandstone beds. The high degree of weathering of this narrow zone of shale and its location suggest a connection between it and the mechanism responsible for causing the shale and sandstone beds down slope to have toppled. It is believed that a single process, also largely responsible for the high degree of weathering of the narrow zone of shale, contributed to causing the topple to occur. The most likely process responsible for the severe weathering of the narrow zone of shale and the toppling of the intact shale and sandstone beds, located down slope, is frost heave associated with ice lens formations within the narrow shale zone. Bell (1992) states that alternate freeze-thaw cycles can cause significant weathering to occur in rocks, a process called congelifraction. Tang (1986) suggested that freeze-thaw cycles were likely the dominant mechanism of toppling in the Highwood Pass area. Using the results from several of the laboratory tests described above, it was confirmed that the weathered shale zone is highly susceptible to frost heave and ice lens formations. Frost heave and ice lens formations in the narrow zone of completely weathered shale could have easily pushed the weaker shale and sandstone beds, located down slope, forcing them to topple and eventually causing them to rupture at depth. Although probably a more active process in the past, frost heaving of the weathered shale is likely still an on going process today. The mechanism of toppling just described will be discussed in greater detail in the following chapter as part of the slope stability analysis of *Rock Glacier Site*.

CHAPTER 7

Stability Analysis

7.1 Introduction

This chapter examines the stability of the slope at *Rock Glacier Site*. The water pressures in the slope are approximated, based on observations made while in the field and on climatic data pertaining to the area. The toppling movements located on the slope are examined in detail and the applicability of analyzing the topples with Goodman and Bray's (1976) numerical procedure is discussed. The process of rock falls is discussed and the importance of rock fall with respect to the history of movements at *Rock Glacier Site* has been outlined. The processes which form rupture surfaces at this site are explained and the significance of rupture surfaces in terms of the overall stability of the slope are examined. A relationship between the amount of angular rotation of a toppling rock mass and the formation of rupture surfaces is investigated. Movements along circular and non-circular sliding surfaces are analyzed for the slope and the most probable locations of these surfaces to form are determined by a slope stability program called SLOPE/W (GEO-SLOPE 1993). A simple block sliding analysis is performed on a large mass of toppling rock near the top of *Rock Glacier Site* to determine a minimum value for the angle of friction along the rupture surface. Finally a sequence of processes which led to slope movements at *Rock Glacier Site* is proposed which ties together the findings of previous chapters in this thesis.

7.2 Water Pressures in the Slope

When analyzing the stability of any slope in terms of effective stresses, the water pressures in the slope must be considered. Higher water pressures reduce the effective stresses which in turn leads to a decrease in the stability of the slope.

The slope at *Rock Glacier Site* is comprised of heavily jointed steeply dipping sedimentary rocks. The close spacing of the joints and the fact that the joints are on average open approximately 1 mm suggest a high permeability of the rock mass. Hoek and Bray (1981) proposed a relationship which correlates joint spacing and joint opening with permeability of the entire rock mass. Using this correlation, the permeability of the rock mass is approximated at 1.0 cm/sec, similar to a clean sand. This indicates that the slope is free draining.

Observations made during the field mapping portion of this research support a high permeability of the rock mass. The spring runoff from melting snow and periods of heavy precipitation were both observed to have immediately drained into the rock mass and then into the gullies where it is channeled down the slopes ending in Pocatera Creek. However, in the earliest stages of field research, the rapid drainage was observed to be impeded by ice and the frozen ground in the base of the gullies. Since the gullies are so deeply cut, they remain shaded for most of each day which delayed the thawing of the ground and the melting of the ice in the lower gullies to well into the month of July. The climatic data also indicates the highest precipitation to be in the spring and early summer months (Environment Canada 1993). This higher precipitation coupled with melting snow and impeded drainage is expected to create an elevated groundwater table in the slope during the spring and early summer months of each year (Nyberg 1985).

The height of the elevated groundwater table during the spring and early summer months can only be accurately determined with field measurements. However, for the purpose of this study, the location of the elevated groundwater table was estimated to explore its effect on the stability of the slope. Using the profiles of each gully, obtained during field mapping (Fig. 4.8), the groundwater table was estimated at roughly 15 m above the base of the gullies. This is an upper bound estimate, to ascertain its effect on the stability of the slope.

7.3 Toppling Movements (Goodman and Bray's Numerical Procedure)

Large portions of the rock mass at *Rock Glacier Site* are toppling. These toppling movements are directly and indirectly responsible for a large percentage of the material accumulated at the base of the slope. The kinematics of toppling movements have been examined in detail by Goodman and Bray (1976). This section examines the applicability of using Goodman and Bray's (1976) numerical procedure to analyze the topples at *Rock Glacier Site.*

Goodman and Bray (1976) divided topples into three separate classes. The first class is flexural toppling which occurs when continuous rock columns bend forward. The second class is block toppling which occurs when individual rock columns, which are divided by widely spaced joints, are rotated forward. The third class of topples is block-flexure toppling which is characterized by the rotation of jointed pseudo-continuous rock columns. The second and third classes of topples are found at *Rock Glacier Site.* However, the classification scheme used to describe the topples at *Rock Glacier Site* follows the one used by Hu (1991). Descriptions of these topples and examples are discussed in section 4.4. Goodman and Bray (1976) also went on to classify some topples as being secondary in nature. Secondary topples, as they called them, occur as a result of other movements which directly cause toppling to develop in areas where they might not otherwise occur. An example of a movement leading to toppling is a slide of material transferring load directly to rock beds which respond by toppling. This same form of toppling movement is referred to as a complex slide - topple by the International Geotechnical Societies' UNESCO Working Party on World Landslide Inventory (WP/WLI 1993). As described in chapter six, frost heaving of the weathered shale zone is believed to be exerting a lateral force on the interbedded shales and sandstones causing them to topple. However, this cannot be interpreted as a secondary topple by Goodman and Bray's (1976) classification because the transferring of load must be caused by some form of slope movement which frost heaving is not.

Goodman and Bray's (1976) kinematic criterion for analyzing topples is based on a formal mathematical solution to a simple toppling problem. This problem of toppling is essentially a two-dimensional phenomenon which can be modeled in two dimensions adequately. The analysis developed by Goodman and Bray (1976) relies heavily on a simplified geometry of the toppling mass. When a rock mass is heavily broken and the orientations of columns are varied throughout the mass, the geometry of the problem must be simplified in order to proceed with the analysis. Goodman and Bray's (1976) numerical procedure seeks to determine the portion of the rock mass which will move by sliding and the portion which will move by toppling. The analysis also provides for a factor of safety to be calculated based on a supporting force applied to the toe block of the rock mass. The analysis considers the inter-block forces between the columns of rock and determines how these forces control the movements of the rock mass.

The rock mass selected for analysis at *Rock Glacier Site* is labeled as "B" on Figure 3.2. This rock mass was selected because it has a well defined rupture surface and the dimensions of the rock mass were easily measured in the field. The geometry of the rock mass is heavily broken and is comprised of zones of rock with different orientations. A cross section of the rock mass being examined is illustrated in Figure 7.1. Some of the material is essentially loose debris in completely random orientations. The overall geometry of the mass can be simplified into columns of rock with the same orientation for analysis but it is the geometry of the rock mass itself which makes this and other topples at *Rock Glacier Site* not suitable for analysis by Goodman and Bray's (1976) numerical procedure. The reason topples at this site are not suitable for analysis is that the rock mass has already toppled and if it slides will likely do so as an intact mass which can be modeled more accurately as a block slide. This same rock mass will be analyzed as an intact sliding block in section 7.6. There are several other difficulties with analyzing the topples at *Rock Glacier Site* with Goodman and Bray's (1976) numerical procedure because they did not contemplate topples on underdip cataclinal slopes. These difficulties will be discussed in the following paragraph.

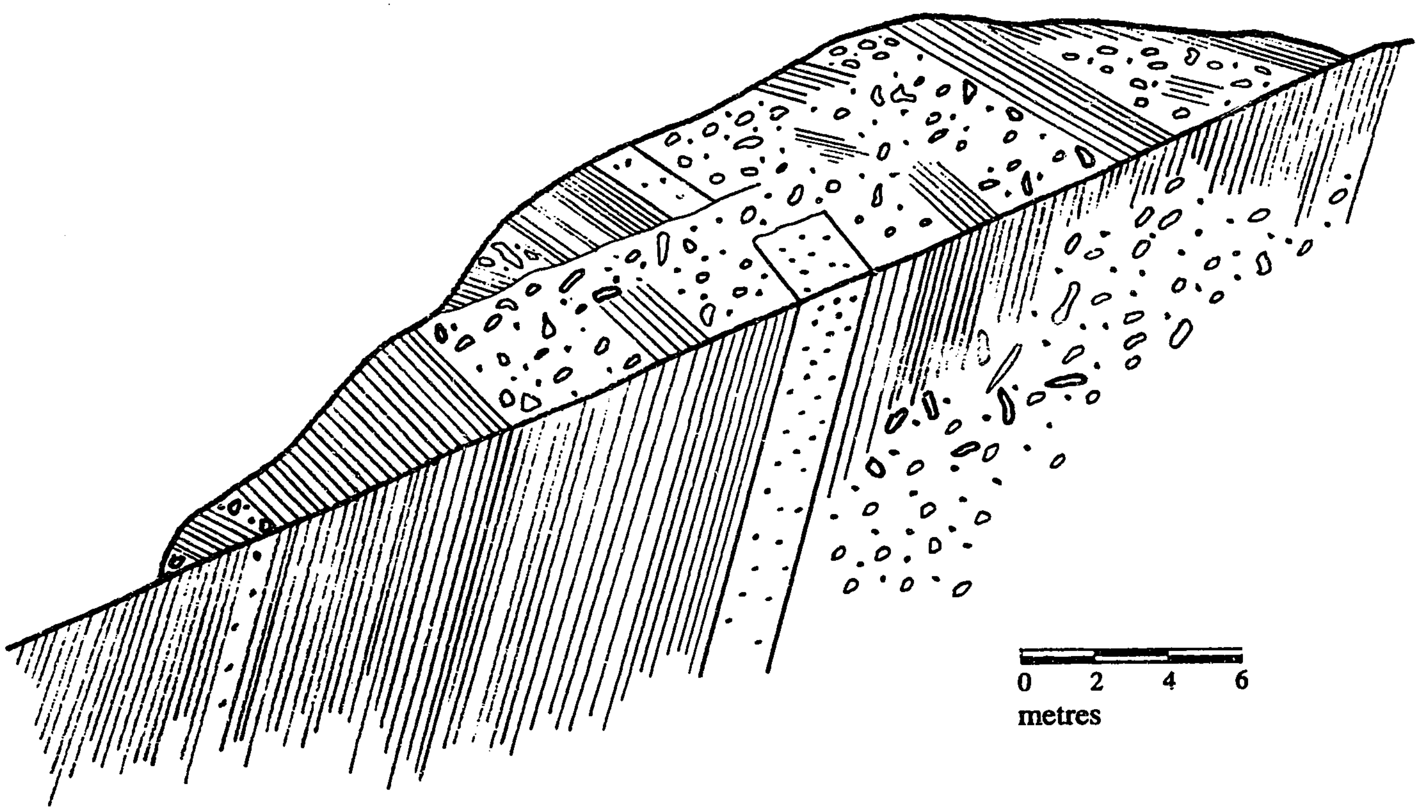


Figure 7.1: Cross-section through topple being examined (labeled as "B" on Fig. 3.2).

There are inherent problems with applying Goodman and Bray's (1976) analysis of topples to underdip cataclinal slopes. On underdip cataclinal slopes, all blocks must start by toppling because the sliding surfaces dip into the slope. As mentioned above, block slides of toppling rock masses are likely to occur once toppling has reached a certain stage of development. In Goodman and Bray's (1976) analysis there is no provision for a block which is initially toppling to change to a block which is sliding. In order to numerically analyze toppling and then sliding on underdip cataclinal slopes, the analysis must allow for or consider the rotation of the sliding surfaces. Goodman and Bray's (1976) numerical procedure for analyzing topples does not consider the rotation of sliding surfaces and does not consider the effects of polishing or wear of sliding surfaces as a result of toppling. The alteration of sliding surfaces, or rupture surfaces, formed as a result of toppling will be discussed in detail in section 7.5. As a result of these problems with applying Goodman and Bray's (1976) numerical procedure to topples on underdip cataclinal slopes, more emphases will be placed on analyzing rock falls and sliding type movements in the following sections.

7.4 Rock falls

Rock falls are characterized by the detachment of rock fragments which then travel down the slope and accumulate at the base. Rock falls and the formation of large talus slopes as a result of rock falls are a common process in mountainous regions (Church et al. 1979). During field mapping at *Rock Glacier Site*, small rock falls were commonly observed. As suggested by Evans and Hungr (1993), these rock falls frequently started out as large blocks that then disintegrated during the process of bouncing, rolling, or sliding down the slope. This section examines the processes controlling rock falls at *Rock Glacier Site* and the role that rock falls have played in contributing to the large mass of colluvium at the toe of the slope. The impact or risk of rock falls will also be assessed in terms of the significance of this process as it relates to the future stability of the slope at this site.

The processes controlling rock falls at *Rock Glacier Site* are governed by the geology of the area and the toppling process. The fact that the sandstones and shales are both highly fractured allows rock blocks to break away easily from the slope. The force which drives rock falls is essentially gravity. However, rock blocks or rock fragments can remain stationary on slopes until either some external force starts them in motion or forces such as cohesion, which hold the rock block or rock fragment back, are broken or overcome. When blocks of loose or already weathered material are displaced, the process is referred to as "secondary rock falls" by Gardner (1980). As discussed in chapter three, weathering is a moderately active process at *Rock Glacier Site*. The process of weathering results in the destruction of cohesion within the rock mass. Thus, a rock block which is prone to falling, but held in place by cohesive forces, will eventually fall after being exposed to the effects of weathering for a prolonged period of time.

Some rock blocks are stable even with no cohesion. These blocks need either an external force to initiate the rock fall or a reorientation of the rock mass to create an unstable position for the block. The latter option is taking place at *Rock Glacier Site* in the form of toppling. Toppling of the rock mass creates a steepened and less stable slope. Ultimately, if the toppling process continues, the entire rock mass will begin to break apart and fall under the influence of gravity. A force which can initiate rock falls is frost action. Schuster and Krizek (1978) point out that frost action is likely to either directly or indirectly account for more rock falls than all other factors combined. In this case, as described in the previous chapter, frost action plays predominantly an indirect role in initiating rock falls by driving the toppling process which steepens the slope.

An important question to ask when examining the large mass of colluvium at the toe of the slope is: what was the dominant process responsible for the large movements which have occurred in the past at *Rock Glacier Site*? Examination of the geometry of the colluvium at the toe of the slope, suggests that rock falls have played a dominant role in its creation. Gardner (1980) also found that low magnitude / high frequency type events, such as rock falls, were the primary contributors to the debris slopes in Highwood Pass. Evans and Hungr (1993) described rock fall talus as being steepened to the angle of repose by the continuous deposition of material and to have a marked reduction in the slope angle near the toe. They described a typical rock fall dominated talus sloped at 38 degrees and reduced at the toe to 10 - 20 degrees. The large mass of colluvium at the toe of *Rock Glacier Site* is at its angle of repose, 40 degrees, and exhibits a zone near the toe of the slope which is significantly less steep, 20 degrees, as described in chapter four.

Evans and Hungr (1993) also determined an empirically based rock fall shadow angle of 27.5 degrees for rock fall slopes. The rock fall shadow angle is the angle between the outer margin of the shadow and the apex of the talus slope, measured from the horizontal (Evans and Hungr 1993). The outer margin of the shadow being defined as the most distant point of rest for boulders displaced by rock falls. At *Rock Glacier Site*, the rock fall shadow angle is 31 degrees. However, the rock fall shadow angle would be closer to the 27.5 degrees found by Evans and Hungr (1993), but the deep ditch constructed up slope of Highway 40 is trapping boulders that would otherwise have traveled slightly further, boulders pre-dating the construction of the highway can be found on the opposite side of Highway 40.

Evans and Hungr (1993) also described typical rock fall talus to exhibit a grading characteristic. The large mass of colluvium at the toe of *Rock Glacier Site* also exhibits this same grading characteristic. At the top of the large mass of colluvium, the fragment sizes of rock debris are significantly smaller than the fragment sizes of debris at the toe of the slope. On average, the size of rocks located at the top of the colluvium was typically less than 0.1 m^3 while at the toe of the slope the size of the rocks was usually less than 0.5 m^3. All of these features outlined above support the idea of rock falls having played a dominant role in forming the large mass of colluvium at the toe of *Rock Glacier Site*.

Rock falls at *Rock Glacier Site* are important because they result in debris accumulating at the toe of the slope. However, they are not of great concern in terms of stability because each fall is small and the transfer of material is not destabilizing. The rock blocks found at the toe of the slope are typically less than 1.0 m^3 in size. There also appears to be little risk of boulders rolling or bouncing onto the highway. As mentioned above, the steep ditch appears to capture all boulders which reach it and there is no evidence of scarring on the highway. There is a concern related to boulders being entrained in snow avalanches, slush avalanches, or debris flows which can pass over Highway 40, but these events are more or less restricted to the winter and spring months when Highway 40 is closed through Highwood Pass.

7.5 Rupture Surfaces

Hu's (1991) research in Kananaskis Country found that large volumes of rock masses can be displaced by sliding along rupture surfaces. These near planar surfaces develop between toppled and untoppled rock masses. Rupture surfaces form as a direct result of the toppling process. As a rock mass topples, initially the rock experiences flexure, but, at some point a rupture surface will form at depth within the rock mass (Hu 1991). The rupture surface develops through rotational shearing of the steeply dipping beds. It is a progressive development similar to that of a shear zone in clay slopes. This section will examine the rupture surfaces found at *Rock Glacier Site* in detail and determine empirically the amount of angular shear that develops before a rupture surface forms. The friction angle acting along these rupture surfaces will also be discussed in some detail.

A total of 20 topples were identified at *Rock Glacier Site*. Sixteen of these topples resulted in the formation of a rupture surface. One large rupture surface, identified in the north gully, could be traced for approximately 200 m in length. A large part of the rupture surface was covered in debris and could not be directly observed. The remaining 15 rupture surfaces were small, ranging between 3 to 20 m in length. Table

7.1 summarizes the information for each of the 20 topples found at *Rock Glacier Site*. The dip angle of rupture surfaces varied between 20 and 30 degrees. In most cases, rupture surfaces were not perfectly planar. On average, the dip angle varied between two and five degrees along its length. The question marks after the block flexure topples listed in Table 7.1 are related to the fact that these topples are difficult to recognize when there is very little angular rotation within the entire rock mass. The two topples with no h/b ratio listed in Table 7.1 were a result of problems with measuring either bedding thickness or joint spacing at these locations which were highly weathered. Figure 3.2 shows the locations on the airphoto of all 20 topples listed in Table 7.1. Two representative rupture surfaces at *Rock Glacier Site* will be discussed in greater detail as examples.

The first example of a rupture surface at *Rock Glacier Site* is shown in Figure 7.2a. This rupture surface is located in the north gully and is marked as location "D" on Figure 3.2 and in Table 7.1. The rupture surface is located in sandstone and interbedded shale beds. At this location there is a well defined rupture surface where the beds have rotated 33 degrees. Above the rupture surf [illegible] inue to have rotated but not by more than 10 degrees between adjace [illegible] ng to Cruden and Hu (1994) this topple is classified as a block to [illegible] tion along the rupture surface was possible at this topple [illegible] was possible to reach into the rupture surface to depths close [illegible] rface it felt jagged but free of loose material. There was n [illegible] ons or moisture along the entire length of this rupture surface. [illegible] is observation is likely only to be applicable near the edges where rain and melt water can keep it clean.

The second example of a rupture surface is shown in Figure 7.2b. This rupture surface is located in the upper south gully and is marked as location "E" on Figure 3.2 and in Table 7.1. This rupture surface is located in shale and interbedded sandstone beds. At this location the rupture surface is once again clearly defined, however, it was not open and could not be observed in any detail similar to the first example. This